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UNITED STATES DEPARTMENT OF AGRICULTURE
BULLETIN No. 852
Contribution from the Bureau of Public Roads
Thos. H. MacDonald, Chief

Washington, D. C. PROFESSIONAL PAPER October 28, 1920

THE FLOW OF WATER IN CONCRETE PIPE

By

FRED. C. SCOBEY, Senior Irrigation Engineer

With Discussion by KENNETH ALLEN, ARTHUR S. BENT, F. C. FINKLE, ALLEN HAZEN, J. B. LIPPINCOTT, and H. D. NEWELL

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WASHINGTON
GOVERNMENT PRINTING OFFICE
1920

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INTRODUCTION.

Consequent to the general increase in use of concrete during the past 15 years, it has been but natural that this material should be tried for pipe lines; and, although there are localities where poor construction has brought concrete pipe into disfavor, where properly made it is undoubtedly a success. For a long time concrete was thought adaptable to low heads only. Now it is used extensively in this country for heads exceeding even 100 feet, and in Europe lines have been constructed to withstand pressure heads of several hundred feet. Where the heads exceed more than 15 or 20 feet the pipe is, as a rule, reinforced with steel.

The fact that it may be made at or near the place of use recommends this type of construction for irrigation or other pipes far removed from railway lines. In many cases the forms and cement

Note.—This bulletin treats of the subject of flowing water in concrete pipes. It is based on field tests made on pipes in commercial operation. This publication is offered for use of engineers designing and measuring concrete pipes for irrigation, power, municipal, mining, or other purposes and for courts and attorneys at law interested in cases involving the carrying capacities of concrete pipes.

are the only items requiring an extended haul. Its cost, compared with that of cast iron and steel, its permanence and comparatively fireproof qualities, are in its favor. When reinforced it is usually more costly than wood-stave pipe, and has not the required elasticity of wood or metal when subjected to a wide range of temperatures or to water hammer. Proper expansion joints will care for temperature changes.

Concrete pipe has usually been considered inferior to wood pipe in carrying capacity and the tests described in this paper confirm this belief as regards average wood pipes and average concrete pipes. These tests also show that the highest grade concrete pipe will convey slightly more water than can safely be assumed for the same size of wood-stave pipe. The results of a study of the capacity of wood-stave pipes were embodied in a previous paper.[1]

While most of the experiments described in this bulletin were made on pipes under pressure, there are some tests on record which were made on flow-line pipes and conduits running to partial capacity; that is, with the surface exposed to the air. The experiments made under these two conditions will be taken up separately, in parts 1 and 2 of this paper.

NOMENCLATURE.

Unless otherwise noted, the various symbols used throughout this publication will have the following significance:

d—The mean inside diameter of the pipe in inches.
D—The mean inside diameter of the pipe in feet.
r—The mean inside radius of the pipe, or $\frac{1}{2}D$, in feet.
Q—The mean discharge of the pipe, during the test, in second-feet.
A—The mean area of the pipe bore, in square feet, $=\pi r^2$.
V—The mean velocity of the water, during the test, in feet per second.
L—The length of reach tested, in feet.
h_f—The head of elevation lost in overcoming internal resistance within a fairly straight pipe of uniform size, in feet, $=\frac{HL}{1{,}000}$.
H—The above loss (termed friction loss) per 1,000 linear feet of pipe $=\frac{1{,}000\,h_f}{L}$.
h_v—The head of elevation lost in creating the mean velocity, V, in feet. Called velocity head.
h'_v—The velocity head recovered as the velocity is reduced at the pipe outlet, in feet.
h_e—The head of elevation lost at a pipe intake due to impact and entrance resistances, in feet, here called entry head.
P—The wetted perimeter; in a pipe under pressure, the inside circumference, $=\pi D$ or $2\pi r$, in feet.
R—The hydraulic radius $=\frac{A}{P}$; in a circular pipe, under pressure, $=\frac{D}{4}$.
s—The hydraulic grade or slope, in feet per foot of length of a pipe of uniform size, $\frac{h}{L}$.
C—The coefficient of retardation in Chezy's formula.
n—The coefficient of retardation in Kutter's formula.
n^1—The coefficient of retardation in Manning's formula.
m—The coefficient of retardation in Bazin's (1897) formula.

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[1] "The Flow of Water in Wood-stave Pipe," by Fred C. Scobey, Bulletin 376, U. S. Department of Agriculture.

C_w—The coefficient of retardation in the Williams-Hazen formula. (Not to be confused with C, C_m, or C_s.)

C_m—The coefficient of retardation in Moritz's formula. (Not to be confused with C, C_w, or C_s.)

C_s—The coefficient of retardation in the new formula. (Not to be confused with C, C_w, or C_m.)

f—The coefficient of retardation in the Weisbach formula. (This formula is variously known as Weisbach's, Weston's, Darcy's, and Chezy's.)

K—The coefficient of d in the general exponential formula for flow of water in pipes (formula 14, p. 45).

Cement pipe.—Composed of Portland cement and sand or fine gravel, without coarse aggregate.

Concrete pipe.—Composed of Portland cement, sand, and coarse gravel or broken stone. Will also be used as the broad term applying to both kinds of pipes for the reason that, in reality, the so-called "cement" pipe is concrete.

Number.—Wherever a pipe number is given, the reference is to the corresponding number in Tables 3, 4, and 11, and to the description of the pipe under that number.

TYPES OF PIPE.

In so far as the bounding material and other elements that influence the carrying capacity are concerned there are three distinct types of pipe, as follows:

1. Composed of an assembled length of precast units.
2. Constructed-in-place or monolithic pipe.
3. Cement-lined metal pipe.

Types 1 and 2 subdivide into three kinds:

a. Interior surface as left by the forms.
b. Interior surface washed with cement mortar, neat cement grout, or asphalt.
c. Interior surface improved by troweling the joints and generally "pointing up" rough places or by rubbing down irregularities with an abrasive. (See p. 83.)

Pipes of type 1 are made with either a "dry mix" or a "wet mix." A dry mix, as its name implies, is made with a minimum of water. This permits the immediate removal of the inner and outer molds, the pipe section being cured while resting on a cast-iron base ring. This ring holds the socket end of the section true to shape, but the bevel end is generally distorted to a slight extent during the removal of the forms. The distortion is never sufficient to impair the sectional area if the inner form is true to nominal size, but all distortion is manifest in the assembled pipe line by offsets or shoulders which cause loss of head.

When tamped by hand an interior coating of cement mortar or asphalt is necessary to make pipe of this type impervious to leakage, but when the pipe is made in a machine the power tamper renders the concrete sufficiently dense without a coating to withstand the passage of water under ordinary pressure.

The cost of the many forms necessary in the manufacture of wet-mix pipe precludes the use of this method except for the highest class of work. The time of setting is greatly hastened by the use of steam. The greatest carrying capacity is attained with pipes made in this manner. One maker in California has succeeded in using a mixture so damp that coating is unnecessary, yet the forms are removed immediately, as though the pipe were made with a dry mixture. Before setting, this pipe slumps down slightly, contributing to distortion of shape and thickening of the shell. The "pull" of the forms being removed before setting leaves a rough interior surface.

Pipes made with a very wet mixture must of necessity be left in the forms until well set. Oiling or soaping the forms is necessary to prevent sticking, and makes for a smooth interior surface if the mixture is well worked when put into the forms. Also, if the forms are rigidly true to shape, the resulting pipe is not distorted.

In the assembling of precast units there must of necessity be a joint every few feet. The standard length of small pipes is from 2 to 3 feet, while large ones are made in sections up to 8 or 10 feet long. In the early days extreme care was not taken to make smooth joints, but in present-day construction of the best workmanship on lines with pipe of such a size that a man may work inside, the joints can hardly be distinguished from the rest of the pipe, except by the color. On smaller pipes the joints are carefully wiped, so that all excess mortar is removed.

At first thought it would seem that a pipe of monolithic construction should present an unbroken surface, but in actual practice it is a difficult matter to move the forms forward so that shoulders or offsets are not formed in the pipe at the ends of the sections. (See Pl. II, fig. 2.) This is especially true on curves. These shoulders may be much improved by touching up the interior after removal of the forms, but in many cases this does not appear to have been done. In retouching, the general idea should be to avoid sudden enlargements or contractions and all shoulders should be tapered to minimize loss of head.

Wood forms are of course cheaper than steel, but the resulting surface is very inferior unless most carefully made, as any crack between the boards is clearly defined in the concrete. This may be bettered by a surface coat, but it is doubtful if any coat is as smooth as the surface first left by a well-oiled, rigid steel form, if the concrete is carefully worked into close contact at all points. Coating wooden forms with sheet metal of course gives approximately the same surface as steel forms.

PART I. FLOW IN PRESSURE PIPES.

FORMULAS FOR FLOW OF WATER IN CONCRETE PRESSURE PIPE.

Water is caused to flow and velocity created by the force of gravity. Thus the flow follows the general law of falling bodies, and the velocity tends to become constantly accelerated. This tendency is just balanced by the influences retarding the flow.

For a pipe carrying flowing water under pressure, the difference in elevation H_E (fig. 1), between the surfaces of the water at the intake and outlet is the effective head through which the force of gravity acts. The effective or lost head is made up of several individual losses as follows (fig. 1):

$$\text{Velocity head} = h_v = \frac{V^2}{2g} \tag{1}$$

This is the head absorbed in creating the mean velocity V, at which the water is conveyed through the pipe. This loss occurs at the intake. As a rule, little or none of this velocity head is recovered at the outlet of the pipe.

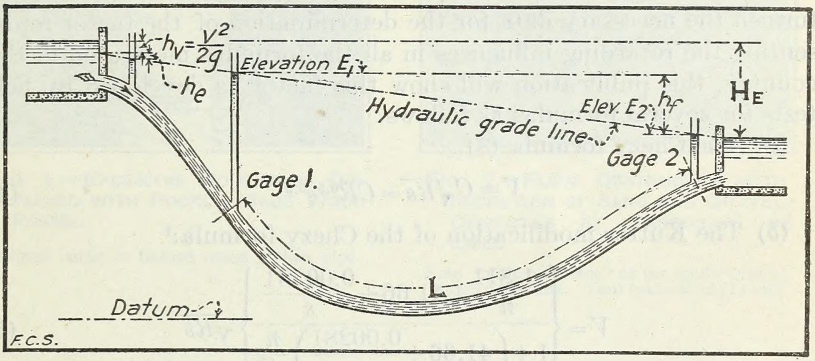

FIG. 1.—Hydraulic elements for loss of head in siphon pipe.

$$\text{Entry head, } h_e = \frac{h_v}{2} \text{ (approximately)} \tag{2}$$

The amount of loss at the entry, due to the effect of contraction eddies and other retarding influences, is variable and uncertain, but most authorities agree that it should be taken as half the velocity head, unless the inlet structure is especially designed to minimize this loss. For further discussion see page 52.

Friction head, h_f, is that lost in overcoming the retarding influences within a reasonably straight pipe. In pipes of great length, the amount of this loss so far exceeds the two losses first mentioned that they often may be neglected, especially in small pipes. This is the loss upon which the experiments described in this paper were concentrated.

In addition to the above losses, there may be others, such as those due to bends and valves or other obstructions; but as a general thing, these items do not enter the design of concrete pipes, especially for irrigation purposes. For this purpose the pipe is laid on such gentle curves, both horizontal and vertical, that such losses need not be considered.

In 1775, Chezy, a French engineer, offered his now well-known formula for the flow of water in both open channels and closed conduits:

$$V = C\sqrt{Rs} \tag{3}$$

Here C is a coefficient, originally thought to be constant, but now known to vary with functions of the slope, the hydraulic radius, the velocity, and with some factor representing the retarding influences in the channel. Some of the formulas used in this country for the design of pipes have accepted the Chezy formula as a basis and made only such modifications as experience dictated.

Since the hydraulic elements secured in the field experiments furnish the necessary data for the determination of the factor representing the retarding influences in all the formulas most used in this country, this publication will show this factor as developed by field tests for several formulas as follows:

(*a*) The Chezy formula (3).

$$V = C\sqrt{Rs} = CR^{0.5}s^{0.5} \tag{4}$$

(*b*) The Kutter modification of the Chezy formula:[1]

$$V = \left\{ \frac{\frac{1.811}{n} + 41.66 + \frac{0.00281}{s}}{1 + \left(41.66 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{R}}} \right\} \sqrt{Rs} \tag{5}$$

in which C is elaborated so that it takes into consideration the influence of the hydraulic grade and of the mean hydraulic radius and introduces a new variable, n, which is supposed to represent all the retarding influences.

(*c*) The Weisbach formula, which has been used by textbooks as a general formula for flow of water in clean pipes:

$$h_f = f\frac{LV^2}{2Dg} \tag{6}$$

(*d*) The Williams-Hazen general formula[2] for many kinds of pipes:

$$V = C_w R^{0.63} s^{0.54} 0.001^{-0.04} \tag{7}$$

---

[1] E. Ganguillet and W. R. Kutter, translated by Rudolph Hering and John C. Trautwine, jr. A General Formula for the Uniform Flow of Water in Rivers and other Channels, New York, 1907, 2d ed.

[2] Hydraulic Tables, Williams and Hazen, 2d ed., New York, 1909, p. 1.

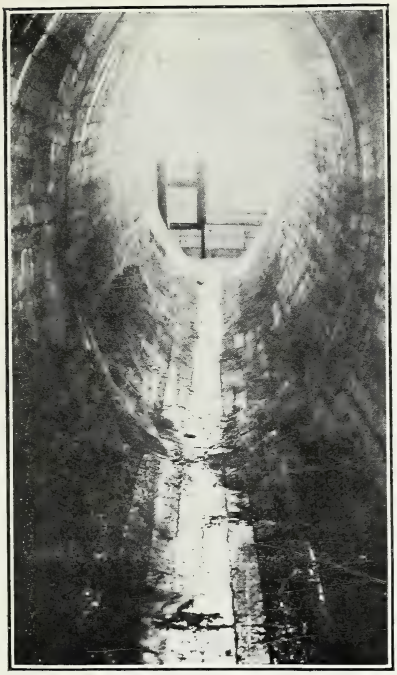

FIG. 1.—EXCESSIVE ROUGHNESS OBTAINED WITH POORLY MADE WOOD FORMS.

Ellipsed outlet to 60-inch round siphon pipe.

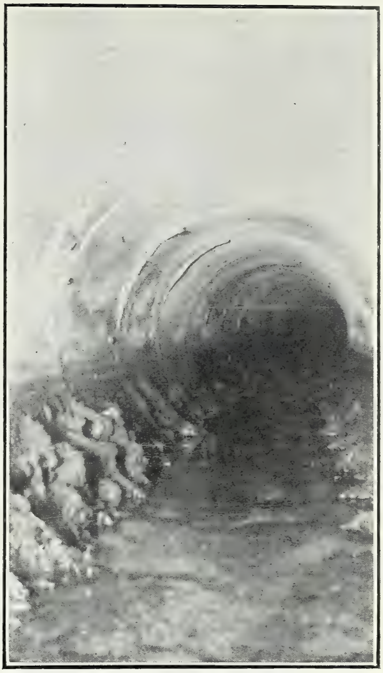

FIG. 2.—FLOW OBSTRUCTED WITH ACCRETION OF SAND AND GRAVEL, CEMENTED BY CARBONATE OF LIME.

Note "mortar squeeze" at the joints; typical of very old lines. View taken after 14 years' use.

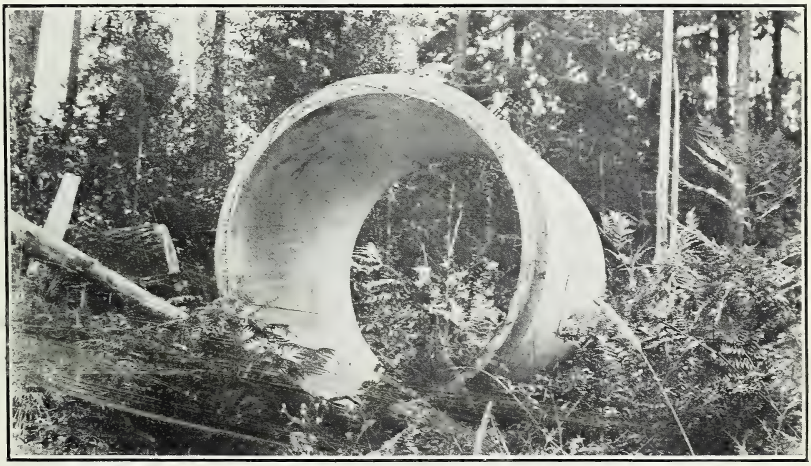

FIG. 3.—PIPE UNIT FOR VICTORIA AQUEDUCT, BRITISH COLUMBIA.

Exceeding smoothness obtained with oiled steel forms and omission of any wash coat.

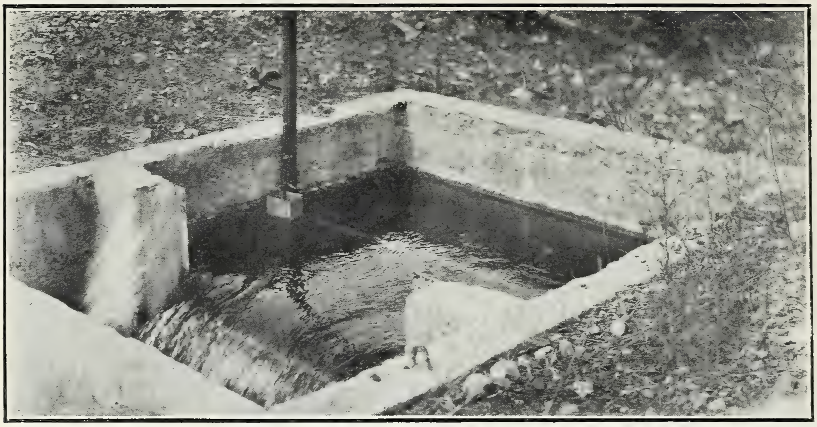

FIG. 1.—WEIR AT INTAKE OF BISHOP LATERAL, BRITISH COLUMBIA FRUITLANDS CO., KAMLOOPS, BRITISH COLUMBIA (NO. 17).

Note hook gauge and stilling box.

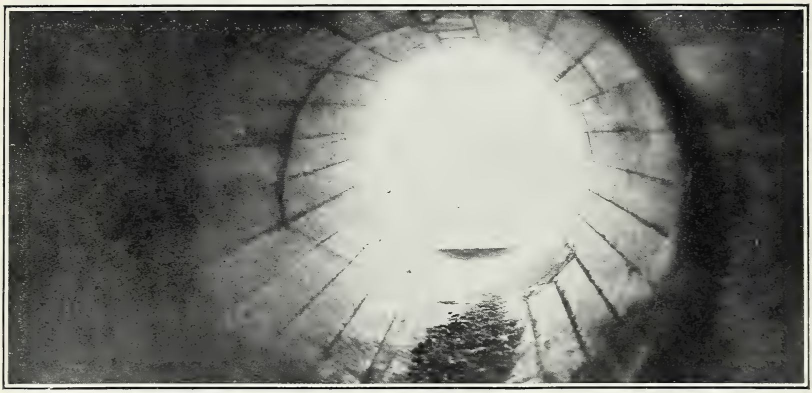

FIG. 2.—OUTLET OF CLAVEY SIPHON, OAKDALE IRRIGATION DISTRICT, CALIFORNIA.

Note ridges left by imperfect wood forms, also hardened scraps of concrete stuck to bottom, also belting ridges at ends of forms. (Pipe No. 20.)

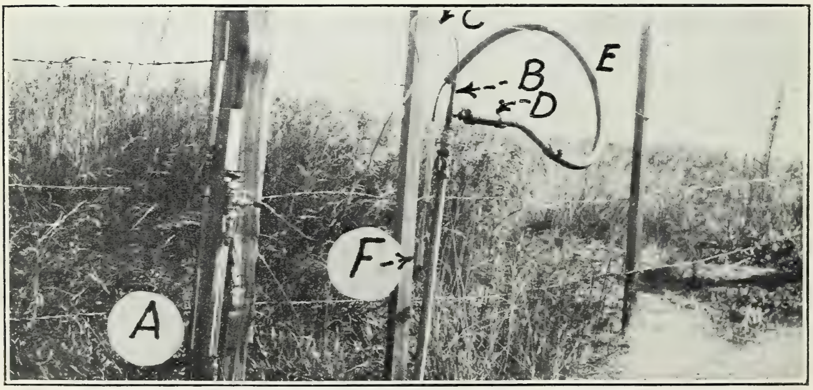

FIG. 3.—GAUGE ON D_1 LINE, UMATILLA PROJECT, U. S. RECLAMATION SERVICE, OREGON. (PIPE NO. 23.)

Shows mercury manometer (A); stuffing box (B); piezometer tubing (C); color reservoir (D); color gun (E); and pump (F).

which may be arranged in the same form as formula 10, becoming

$$H = k\frac{V^{1.852}}{D^{1.167}} \tag{8}$$

The authors of the formula suggest the following values of C_w:

> For masonry conduits of concrete or plaster, with very smooth surfaces, when clean, values of $C_w=140$ may be observed. Generally, such surfaces become slime covered, reducing the values of C_w to 130 or less in a moderate length of time; and if the surfaces are only a little less smooth, say in such shape as is represented by ordinarily good work, the value of C_w is reduced to 120.

It is to be remembered that at the time the above recommendation was made there were very few experimental data upon the carrying capacity of cement-plastered and concrete pipes or conduits. However, with all the data now available, the writer does not make any material difference in his recommendation given on page 64.

(*e*) The Moritz formula[1] for "concrete pipe built continuously with steel forms,"

$$Q = 1.31 \; D^{2.7} \quad H^{0.555} \tag{9}$$

or

$$H = \frac{9.11\,V^{1.8}}{d^{1.26}} = \frac{0.40 \; V^{1.8}}{D^{1.26}} \tag{10}$$

Moritz suggests reducing the above coefficient of 1.31 from 5 to 10 per cent for "jointed pipes, depending upon the amount of care used in producing a smooth interior surface."

(*f*) For the reasons set forth on pages 45 to 49 the writer offers the following formula, which differentiates between various classes of concrete pipes by means of a coefficient C_s. This formula is new to the extent of suggested coefficients only.

$$\mathrm{V} = C_s H^{0.5} d^{0.625} \tag{11}$$

$$H = \frac{V^2}{C_s^2 d^{1.25}} \tag{11a}$$

$$Q = 0.00546 \; C_s \; d^{2.625} \; H^{0.5} \tag{11b}$$

in which the following values are suggested for the coefficient C_s:

Class 1. $C_s = 0.267$. For old California cement pipe lines. It appears to have been the general practice throughout southern California during the early eighties to lay the pipes with a generous supply of mortar and make little or no effort to remove the "mortar squeeze" at the joints; hence these pipes, even though perfectly clean, offer great retardation to the flow of water. This coefficient is also recommended for pipes of class 2 used to convey sewage. The present practice is to wipe all joints carefully and the resulting surface approaches that of class 2.

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[1] Working Data for Irrigation Engineers. By E. A. Moritz, New York, 1915, p. 66. Also see Flow of Water in Pipes, E. A. Moritz, Eng. Rec., vol. 68, No. 24, p. 667.

Class 2. $C_s = 0.310$. For modern "dry-mix" concrete pipe and monolithic concrete pipe or tunnel linings made over rough wood forms. Also for surfaces as left by cement gun process. This coefficient should be used for pipes as commonly made at the present time in the west coast States; that is, in 2-foot sections, with a dry mixture, afterwards washed with cement mortar on the inside, the work of manufacture and laying being done by contract under little or no inspection. Under favorable conditions, such as clear water to be conveyed, carefully made joints and thorough inspection, this class may be made to approach class 3, but unless sure of his conditions the designer should use class 2 (for which Table 6 has been prepared). This is especially true for pipes less than 12 inches in diameter, because of the difficulty in making smooth joints.

Class 3. $C_s = 0.345$. For small "wet-mix" pipe in short units; for "dry-mix" pipes in long units; for average monolithic pipe made on steel forms. These pipes may be evenly washed with cement mortar or asphaltum. For small cement-lined iron pipes and for concrete pipe made under pressure with interior coating of neat cement applied by a mechanical "trowel." To be used for pipes of class 4 when the water contains detritus or the line is to be used to convey sewage. (See Tables 7 and 8.)

Class 4. $C_s = 0.370$. For glazed-interior pipe lines; for large cement-lined iron pipes; for monolithic pipe lines where joint scars and all interior surface irregularities are removed. Particularly applicable to jointed lines of units made from wet, well-spaded concrete, deposited against oiled steel forms and allowed to set firmly before forms are stripped. The glazed surface resulting from this treatment is to be untouched with brush or other "wash" process and the units are to be so uniform in shape that no shoulders are perceptible to the touch when the line is finished. The finished joints are to be practitically as smooth as the rest of the pipe. (See Table 9.)

This class covers only the highest grade of workmanship and materials. The specifications are rigid, but have been and can be attained commercially by an experienced organization. They are difficult to attain in a pipe less than 30 inches in diameter—too small to permit a man to work comfortably inside, and are probably prohibitive for sizes less than 12 inches in diameter. A few of the pipes upon which experiments were made appear to have coefficients higher than 0.370, but the writer wishes to be conservative in recommending a coefficient that necessitates a surface so nearly ideal. That is to say, a better surface may be attained in construction than should be anticipated in design.

All the above formulas will be taken up again after an analysis of the data, and specific recommendations made with regard to each one so that the engineer familiar with one type of formula, not desiring to change to a new one, may have the best suggestions the available data offer in terms familiar to him.

OPINIONS OF ENGINEERS REGARDING THE CARRYING CAPACITY OF CONCRETE PIPE.

Experiments upon the flow of water in concrete pipes have been so few in number that there has been developed no clearly defined trend in thought, as was the case with the capacity of wood-stave pipes.

In his discussion of tunnels, Finkle[1] advocates applying a one-fourth inch coat of cement and sand in a 1 to 2 mixture to give a smooth surface and prevent percolation through the main concrete walls. "By means of this construction," he adds, "the coefficient of roughness in the Kutter formula has been reduced as low as 0.010 and in other cases it has been as high as 0.012. If the work is very poorly done it might run to 0.013."

Schuyler states[2] that the conduit supplying Mexico City was designed with a value of n equal to 0.017, but upon examination after construction he remarks, "The interior of the conduit is very smooth, and answers to Kutter's rating of 0.011."

In his conclusions concerning the use of small cement-lined iron pipe, Metcalf has this to say of the carrying capacity:[3]

> Satisfactory data are lacking upon the carrying capacity of cement-lined pipes. It is believed that under favorable conditions the coefficient of discharge to be used in the Hazen and Williams formula is about $C_w=120$, but under actual conditions this coefficient has been found, in several carefully observed cases, to lie between 95 and 110. Unless the conditions are definitely known, therefore, the use of a coefficient not exceeding $C_w=100$ to $C_w=110$ in the Hazen and Williams formula is to be recommended.

Jorgensen used a value of n of 0.012 in the design of a 6-foot reinforced concrete flow line.[4]

Conway[5] says that n was taken as 0.013 in the design of some lines in Mexico. The pipe units were 61 cm. (practically 2 feet) long, made with a "dry mix" and afterwards coated "with a Portland cement grout to which a little freshly slaked lime was added."

TABLE 1.—*Friction factors used in designing concrete pipes of United States Reclamation Service.*

Project.	Line.	Type.	Size.	Kutter's used.	Length.
			Inches.		*Feet.*
Salt River	Pinto Creek	Continuous	63	0.012	2,130
Do	Cottonwood canyon	do	63	0.012	500
Tieton	Various	Jointed	6 to 30	0.013	255,689
Sunnyside	Mabton	do	54		3,100
Do	Prosser	do	30½	0.013	3,088
Umatilla	Various	do	12 to 47		61,728
Belle Fourche	Belle Fourche	Continuous	70	0.012	3,565
Milk River	Willow Creek	do	64	0.014	1,479
Boise	Chance	Jointed	30½	0.013	4,770
Do	Brock	do	30½	0.013	3,546
Do	Forest	do	36	0.013	8,575

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[1] The Most Economic Type of Hydraulic Power Conduits. F. C. Finkle, Eng. Rec., vol. 52, Sept. 2, 1905, p. 263.

[2] The New Water-Works and Reinforced Concrete Conduit for Mexico City. J. D. Schuyler, Eng. News, vol. 55, Apr. 19, 1906, p. 435.

[3] Wrought-Iron Cement Lined Water Pipe. Leonard Metcalf, Eng. News, vol. 61, p. 2, Jan. 7, 1909.

[4] Eng. News, vol. 62, Aug. 5, 1909, p. 146.

[5] The Water Works and Sewerage of Monterey, Mexico, by G. R. G. Conway: Trans. Amer. Soc. Civil Engin., vol. 72, 1911, p. 497.

Concerning the capacity of concrete pipes for the conveyance of sewage, Metcalf and Eddy quote from correspondence with various engineers, in substance as follows [1] (in all cases the friction factor is Kutter's n):

J. W. Alvord uses 0.015 for concrete, but has come to the belief after completing a number of miles that he did not secure work in the Chicago district that justified much less than 0.016.

G. G. Earl uses 0.012 to 0.013 for concrete sewers in New Orleans.

G. W. Fuller uses 0.013 for concrete sewers of greater diameter than 24 inches.

J. H. Gregory uses 0.015 for concrete sewer trunk lines in the Passaic Valley.

Hardesty [2] states that a value of n of 0.012 was used in designing a tunnel lining of concrete deposited against wood forms. This statement called forth a letter from C. F. Mullings [3] of the Madras Presidency, India, to the effect that orders were recently given by the Government inspector general of irrigation (India) to use a coefficient of 0.017 in the design of concrete lined tunnels.

Freeman [4] uses a value of n of 0.014 for concrete tunnels, stating:

> This is liberal enough to cover some roughness of finish and to provide against the retarding effect of mosslike growths, such as was noted in the Boston aqueducts, and will allow for retardation by some slight deposits of sand on the bottom, although there will be small chance of this under the high velocities proposed.

Freeman goes on to state that while the linings are new and clean the value of n would probably be 0.011. For concrete-lined steel pipes he suggests a value of 0.013.

In the same report (p. 350) C. E. Grunsky uses 0.013 for a lined tunnel 10 feet in diameter.

The board of consulting engineers for the Winnipeg aqueduct makes the following recommendations: A coefficient of 132.8 (in the Chezy formula) for a concrete section of basket-handle shape, 10 feet wide and 9 feet high, with hydraulic mean radius of 2.33 feet; a coefficient of 124.1 for a section of similar shape 5 feet wide and 5 feet high with a hydraulic radius of 1.24 feet: [5]

E. G. Hopson, in the report to the city of New York of Burr, Hering, and Freeman,[6] recommended a value for C, in the Chezy formula, of 128 for a concrete aqueduct, based on a diminution of 12½ per cent on open-trench portions, due to slimes. For tunnel sections the reduction in capacity by sliming was considered as only 5 per cent, but the inferior workmanship obtained in tunnels would make the final coefficient in both cases 128.

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[1] American Sewerage Practice, Metcalf and Eddy, 1st ed., New York, 1914, vol. 1, p. 97.

[2] W. P. Hardesty in Eng. News, vol. 56, p. 391.

[3] *Id.*, vol. 57, p. 245.

[4] Hetch Hetchy Water Supply for San Francisco, 1912, by Jno. R. Freeman, San Francisco, 1912, p. 221

[5] Canadian Engineer, Oct. 23, 1913, p. 605.

[6] Report of Commission for an Additional Water Supply for the City of New York, New York, p. 214.

The board of consulting engineers[1] who reviewed the plans of the Los Angeles aqueduct suggested a coefficient for cement-lined tunnels of 0.014 for *n* in the Kutter formula. This value was used in the accepted design.[2]

After conducting a series of experiments upon both open and covered channels in southern California (see p. 88 in Appendix) J. B. Lippincott concluded:[3]

> It would appear from these experiments that a coefficient of 0.012 for *n* in the Kutter formula would be safe in tunnels or covered concrete conduits with plastered surfaces.

In correspondence with the writer, under date of June 15, 1915, Mr. Lippincott writes:

> An interesting feature relative to coefficients is the fact that in concrete-lined conduits in the Southwest, the effect of sunlight is very material in determining what the coefficient will be in the conduit. In the covered conduits that are dark there is no growth of vegetable or animal life in this section and our values of *n* are in the neighborhood of 0.010 to 0.012. If, however, the same class of lining is uncovered and exposed to the light of the sun, the coefficients are very much more unfavorable and may run up to 0.018.

RECAPITULATION.

As a broad statement, it would appear that the concrete pipes and conduits of the country have been designed by the use of the Kutter formula, and that concrete has been considered as concrete, little or no differentiation being made due to various degrees of smoothness, regardless of forms, mixtures, or surfacing, although the acquired surface, due to slimes, has been considered.

Values of *n* have been chosen from 0.012 up to 0.017, the reasons for this wide divergence not being quite clear in several cases. In no case has the designing engineer accepted literally the Kutter classification of 0.010 for "neat cement plaster" or 0.011 for "cement mortar one-third sand." It may be well to state here that these values of *n* are the only ones given in many standard lists that appear to apply to concrete and were based on very few data obtained under conditions that were probably more nearly ideal than commonly could be obtained commercially.

The Kutter formula has been particularly popular in the West, while in the Eastern States the Williams-Hazen formula is extensively used alongside that of Kutter.

A study of Tables 3 and 11 and of Plate VI will develop what coefficients may be expected from various methods of construction for varying sizes of pipe and varying velocities. While it is still evident that the Kutter formula should not be used with a given value of *n* for a given interior surface throughout the range of sizes encountered, still the change is not so marked as that found in the study of wood-stave pipes.

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[1] This board consisted of Jno. R. Freeman, J. D. Schuyler, and F. P. Stearns.

[2] Construction of the Los Angeles Aqueduct, Los Angeles, Calif., 1916, p. 81.

[3] Engin. News, June 6, 1907, vol. 57, p. 612.

The Williams-Hazen formula appears to more nearly apply if the value of C_w is chosen as suggested on page 64.

NECESSARY FIELD DATA FOR DETERMINING THE RETARDATION ELEMENTS OF VARIOUS FORMULAS.

A glance at pages 5 to 7 shows that for study of the various formulas the same hydraulic elements must be determined by field tests. These are:

(1) The mean velocity, V, of water in the pipe.

(2) The loss of head, h_f, due to retardation in a section of pipe of uniform size, within a known distance.

(3) The internal size of pipe, D or d.

The above data having been secured, the coefficient of retardation may be computed for each of the various formulas.

MEAN VELOCITY OF WATER.

The velocity of the water flowing in a reach of pipe may be measured in two general ways:

(1) Directly, by timing a given volume of water through a known distance.

(2) Indirectly, by measuring the discharge of the pipe, thus determining the quantity, Q, and solving the equation $V=\frac{Q}{A}$.

Where the velocity is tested by the direct method the error is probably smaller than where the indirect method is used, unless exceptional facilities for complete measurements, including interior diameters, are at hand.

LOSS OF HEAD DUE TO RETARDATION.

Most of the recent experiments on the flow of water in pipes of uniform size have been made with piezometer columns. This was the method used by the writer. If a piezometer (fig. 1) be properly attached to the pipe, the pressure in the latter will support a column of water whose surface is at elevation E_1, on the hydraulic grade line. In the same way the pressure at gauge No. 2 will lift a column to elevation E_2. The difference between these elevations is the head lost, h_f, due to the retarding influences.

INTERNAL SIZE OF PIPE.

The method used in ascertaining the inside cross-sectional area of the pipe is recounted in the description of each test. In some cases several joints of pipe, remaining from construction, were measured and their mean inside cross-sectional areas accepted as the internal sizes of the operated pipes. In other cases the nominal diameter of the pipe was accepted.

As made in local pipe yards along the Pacific coast, the smaller sizes of pipe, say from 6 to 20 inches in diameter, appear to run under size. The writer measured two diameters on many sections of pipe, taken at random in various pipe yards. The results may be summed up as follows:

The average true area of 8-inch pipe was 4.4 per cent less than the nominal; of 10-inch pipe was 2.7 per cent less; of 12-inch pipe was 3.5 per cent less; of 14-inch was 1.3 per cent less; of 18-inch was 1.4 per cent less. Sections of greater diameter than 20 inches appear to run quite true to nominal size.

SCOPE OF THE EXPERIMENTS.

The writer conducted 130 observations on 30 separate pipes, 29 of which ranged from 8 inches to 63½ inches in diameter and one 120 inches in diameter. Seventeen pipes were of the "dry-mix," cement-washed, jointed types; 5 were of the "wet-mix," oiled-form, uncoated jointed type; 3 were constructed in the same manner and then washed with cement; 1 was of the wet-mix, monolithic, steel-form, coated type; 3 were of the wet-mix, monolithic, wood-form, uncoated type; 1 of the same construction, coated. All but two of these pipes were "running full"; that is, under pressure. Mean velocities ranged from less than 1 foot per second to more than 9 feet per second.

From other sources also listed in summary Tables 3, 4, and 11, and briefly described in the Appendix commencing on page 77, descriptions of experiments on pipe up to 18 feet in diameter are abstracted.

EQUIPMENT AND METHODS EMPLOYED FOR COLLECTING AND INTERPRETING FIELD DATA.

With the following exceptions, the equipment and methods used were the same as those employed in the experiments on wood-stave pipe and which were described in Bulletin 376 of the department. For the sake of brevity these descriptions will not be repeated here.

Color injector.—The only practicable method of measuring the velocity of water in some of the pipes tested was by timing the passage of some color or chemical. For comparatively small volumes of clear water a saturate solution (from 5 to 8 per cent in air temperatures) of potassium permanginate was used. This color is rather uncertain, being decomposed by some waters and neutralized by reddish or muddy waters. Where certainty of results is desired, almost regardless of the water, fluorescein is the best chemical and Congo red the second choice, in the knowledge of the writer. At the present time, however, either is practically impossible to obtain.

It is desirable to start the color at a gulp, whether in water under pressure or in a pipe but partly filled. The "color gun" used in the experiments upon wood-stave pipe was improved somewhat. As shown in figure 3, *S* is a reservoir into which the solution is poured, through the coupling *W*. The latter is then closed and the air reservoir X blown up with the bicycle pump until under a pressure much in excess of that existing in the pipe under test. At a definite noted time the color is injected into the pipe through the cock V. The gauge readings were taken simultaneously with the passage of the color through the pipe.

Color siphon.—Where the pipe discharged into a sand or division box (Pl. III, fig. 1), the first appearance of the color could be noted easily at the pipe outlet, even though 6 or 8 feet below the surface, if the water was clear, but it was impossible to determine when the last sight of color reached the end of the pipe, as the pool would then be filled with colored water. To overcome this difficulty a length of garden hose was calibrated as a true siphon. The inlet end of this siphon was then held in the pipe outlet and the last appearance of color was observed at the outlet of the hose. From the calibration it was found that an effective head of 1 foot required four seconds to discharge a volume of water equal to the volume of the interior of the hose siphon. Thus, if the outlet of this siphon was always just 1 foot lower than the water in the outlet chamber, then the last appearance of the color in the concrete pipe was just four seconds before its last appearance at the hose outlet.

Air trap.—Since all water flowing in pipes contains more or less air in solution and sometimes even free bubbles of air, an accumulation of air in the piezometer tubes is to be looked for. In order to insure a water column free from air between the pipe and the gauge glass, the attachment shown in Plate V, figure 1, was developed. A solid water column follows the tubing aaa to the gauge glass, while air bubbles, taking the highest path, pass into bb. Any bubbles that might escape the first glass Y at b are given another opportunity to escape at c. The air chamber d is provided with an air valve. In blowing out the pressure tubing, the valve V is opened wide to create a draft. The air valve at the gauge glass is opened very slightly. A relatively large amount of water escapes at V and a small amount at the gauge glass. By watching the glass Y at b it could be seen that no bubbles passed into the tube aaa, thus assuring a column of solid water.

ATTACHMENT OF PIEZOMETERS.

Probably no small portion of the discrepancy between results of tests on carrying capacity of various kinds of pipe is due to the method of measuring the pressure head. Apparatus of some form must be installed so that the pressure head will sustain a water or mercury column to an elevation corresponding to the true hydraulic gradient over the point of application of the pressure. In order to secure this result, all positive or negative influence of velocity head must be excluded. Herein lies one difficulty in making tests on concrete pipes in commercial service.

There are two general ways in which access to the stream within a pipe may be secured. A small tube like *A* or *B*, figure 4, may be thrust directly into the stream, or a connection set in the shell of the pipe, like *C*. Tubes of type *A* or *B* were smoothly tapered, with the holes passing through the opposite sides of the tube. When a connection was to be made far from the end of the concrete pipe, a hole was tapped in the latter and the tube inserted through a stuffing box. According to White[1] a tube of type *A* not only registers the proper pressure head, without velocity-head influence, in an open channel, but also within a closed pipe. His third conclusion is that Pitot tubes whose constants are unity in open canal ratings will remain unity, whatever the pressure of the liquid. His tubes *M* and *N* are of this type. Our tube of type *A* was, in essentials, the same as White's static tubes of types *M* and *N*. Although our tubes of this type were tapered with some care, Lawrence and Braunworth[2] showed that the taper was of little importance.

It is to be regretted that Mills[3] did not continue his experiments on piezometer connections to cover tubes like *A* and *B*. He did, however, establish the fact that a connection of type *C* registers the proper pressure head.[4]

The use of an orifice in the shell of the pipe, normal to the curve—that is, of type *C*—is justified by F. P. Stearns[5] and later by Desmond Fitzgerald.[6]

Carrying the idea still farther, Marx, Wing, and Hoskins proved that—

> "When the pressure in the given cross section of the pipe everywhere exceeded that of the atmosphere an open piezometer will stand at the same height at whatever point of the cross section it be attached, and whether it communicates with the pipe at one point or several."[7]

In each of the experiments described in this paper, regardless of the auspices under which it was conducted, but one connection was made at each end of the reach tested.

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[1] The Pitot Tube: Its Formula. W. M. White, Journal Assoc. Engin. Socs., 27 (1901), p. 35.

[2] Trans. Amer. Soc. Civ. Engin., vol. 57, p. 273.

[3] Experiments upon Piezometers Used in Hydraulic Investigations. H. F. Mills, Pro. Amer. Acad. Arts and Sci., vol. 14 (Whole Series, 1879), p. 26.

[4] Mills concludes "that with an orifice whose edges are in the plane of the side, and passage normal thereto. the piezometric column will stand neither above nor below the surface of the stream, but will indicate the true height of the surface." He also notes that a very slight variation of the passage from the normal causes an erroneous reading of the piezometer column; that where the orifice projects into the stream the column rises above the surface of the stream when the orifice faces upstream or is at right angles to the stream and the column does not rise to the elevation of the surface in the stream when the orifice faces down stream.

[5] Trans. Amer. Soc. Civ. Engin., 14 (1885), p. 3.

[6] *Id.*, 35 (1896), p. 241.

[7] Trans. Amer. Soc. Civ. Engin., 40 (1898), p. 526.

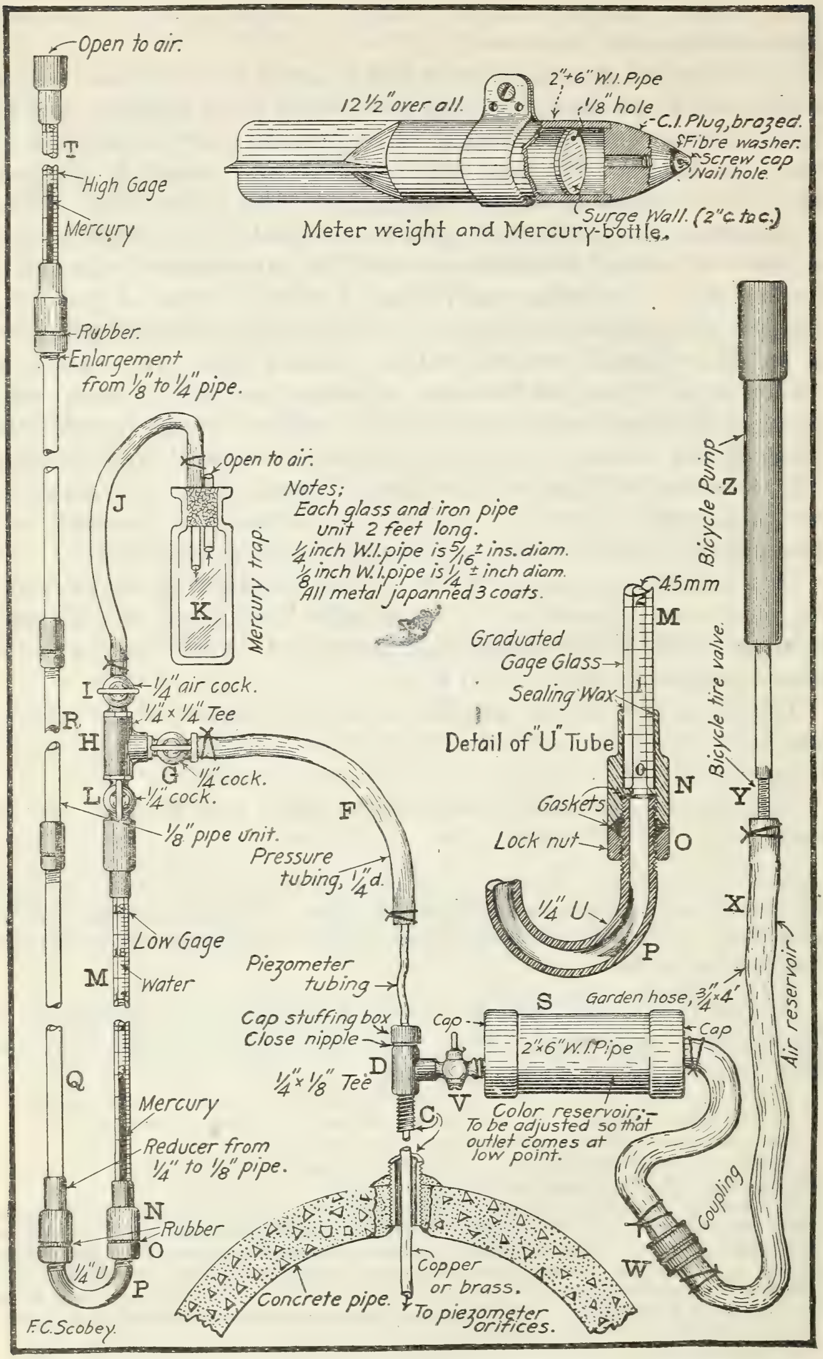
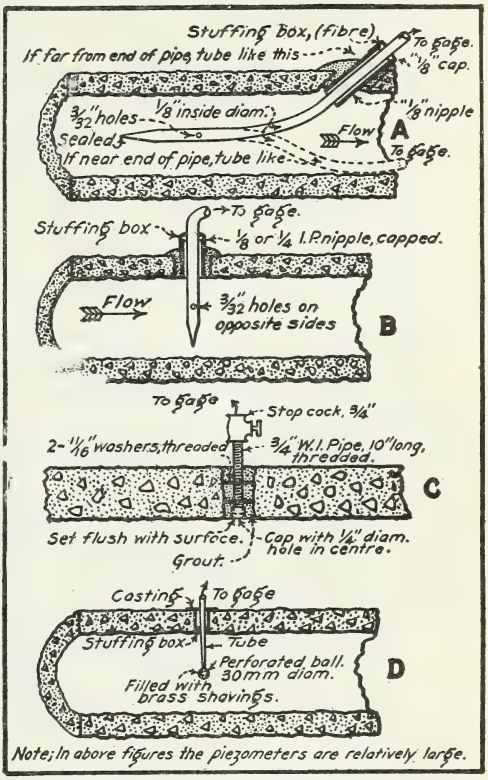


FIG. 2.—Method of attaching mercury manometers to concrete pipes. Details of manometers, fluorescein gun, and combined current meter weight and mercury bottle.

DETERMINATION OF LOST HEAD.

The exact amount of h_f (fig. 1) must be determined. Where a water column is used, say at gauge No. 2, the elevation E_2 is the gauge reading added to the elevation of the gauge zero above an assumed datum, with proper corrections (see p. 5). Where a mercury manometer of the U-tube pattern is used, the reasoning is as follows: It is desired to know the elevation E_1 (fig. 1) for a water column which is the equivalent of a mercury column in a U-tube placed as for gauge No. 1. Referring to figure 2, the mercury in the two legs of the U-tube below Q will be seen to balance. Therefore the pressure of the water at Q is just balanced by the column of mercury QT. But the pressure at c equals that at d. If the mercury X were replaced with water it would reach an elevation sR above Q, where s is the specific gravity of the particular mercury in the gauge, compared with the particular water in the pipe. But the elevation to which this water column would reach is the desired elevation, E_1. As applied to these experiments, referring to figures 1 and 2, the difference in elevation between the readings of the low gauge and the high gauge multiplied by the specific gravity of the mercury and added to the elevation of the low-gauge readings gave the elevation of the equivalent water column when the proper corrections had been applied.


Stuffing box, (fibre)
If far from end of pipe, tube like this
To gage.
1/8" cap.
3/32" holes
1/8" inside diam.
1/8" nipple
Sealed
Flow
A
If near end of pipe, tube like
To gage.
To gage.
Stuffing box
1/8 or 1/4 I.P. nipple, capped.
Flow
3/32" holes on opposite sides
B
To gage
Stop cock, 3/4"
2-1 1/16" washers, threaded
3/4" W.I. Pipe, 10" long, threaded.
C
Set flush with surface.
Cap with 1/4" diam. hole in centre.
Grout.
Casting
To gage
Stuffing box
Tube
Perforated ball. 30mm diam.
Filled with brass shavings.
D

Note; In above figures the piezometers are relatively large.

FIG. 3.—Various arrangements of piezometers for securing pressure head at a given place on the pipe line.

MEASUREMENT OF MEAN VELOCITY.

As a rule, each pipe tested presented its own problem as to the method to be adopted to determine the mean velocity of the water, and in case this method digressed from one of the following standard methods it is described.

Current meter.—Where the water entered or left the pipe in an open channel the discharge was determined with a current meter, and the velocity in the pipe was secured by dividing this discharge by the area of the pipe. The two-tenths and eight-tenths depth method was used, as the results obtained in this way, when compared with the discharge found by the multiple-point method, generally agree with it to about 1 per cent.

TABLE 2.—*Velocities by color compared with velocities by weir and current meter.*

1 Pipe No. (this bul.).	2 Ref. No. (Bul. 376).	3 Pipe diameter.	4 Crest length of weir.	5 Meter method.	6 Velocity per second by color. (V_c).	7 Velocity per second by meter. (V_m).	8 Velocity per second by weir. (V_w).	9 $\frac{V_c - V_m}{V_c}$	10 $\frac{V_c - V_w}{V_c}$
		Ins.	*Feet.*		*Feet.*	*Feet.*	*Feet.*	*Per cent.*	*Per cent.*
	60	8	1.05 Cip.[1]	....	1.251	....	1.295	....	−3.8
	61	8	do	....	1.736	....	1.735	....	.1
	62	8	2.84 Rect.[2]	....	2.048	....	2.09	....	−2.0
	63	8	do	....	3.043	....	2.97	....	2.4
	64	8	do	....	3.294	....	3.37	....	−2.4
	132	18	....	6 tenths[3]	2.08	1.99	....	4.3	....
	192	48	....	2+8[4]	3.14	3.05	....	2.9	....
	193	48	....	do	3.75	3.71	....	1.1	....
	194	48	....	do	4.75	4.73	....	.4	....
	263	78	....	Curve[5]	.911	.928	....	−1.9	....
	264	78	....	do	.963	.974	....	−1.1	....
	265	78	....	do	1.51	1.60	....	−6.0	....
	266	78	....	do	2.063	2.08	....	−1.0	....
	(6)	78	....	do	2.16	2.10	....	2.8	....
	269	78	....	do	2.40	2.37	....	1.2	....
	270	78	....	do	2.44	2.42	....	.8	....
	271	78	....	do	2.79	2.51	....	3.1	....
29	......	36	10.0 Cip.[1]	Int.[7]	3.48	3.48	3.47	.0	.3
29	......	36	do	6 tenths[3]	3.48	3.55	3.47	−2.0	.3
30	......	42	8.02 Rect.[8]	....	1.007	....	.980	....	2.7
30	......	42	do	....	1.19	....	1.147	....	3.6
30	......	42	do	....	1.41	....	1.351	....	4.2
30	......	42	do	....	1.66	....	1.547	....	6.8
30	......	42	do	....	1.75	....	1.734	....	.9
30	......	42	do	....	1.96	....	1.947	....	.7
30	......	42	do	....	2.24	....	2.130	....	4.9
30	......	42	do	....	2.53	....	2.414	....	4.6
30	......	42	do	....	2.91	....	2.769	....	4.8
(9)	......	36	4.99 Rect.[10]	....	1.17	....	1.141	....	2.5
		36	do	....	1.22	....	1.245	....	−2.0
		36	do	....	1.89	....	1.830	....	3.2
		36	do	....	2.69	....	2.619	....	2.6
		36	do	....	2.75	....	2.612	....	5.0
		36	do	....	2.90	....	2.805	....	3.3
		36	do	....	2.99	....	2.935	....	1.8
		36	do	....	3.74	....	3.713	....	.7
		36	do	....	3.80	....	3.669	....	3.4

[1] Cipolletti weir with good conditions of contraction and velocity.
[2] Rectangular weir with end contractions and sharp crest.
[3] Meter held in each vertical at 0.6 depth from surface.
[4] Meter held at 0.2 and 0.8 depths in each vertical; mean accepted for vertical.
[5] Rating curve developed by meter measurements. Velocity taken from curve.
[6] Excluded from Table 2, Bul. 376, because gauge data lost for manometer No. 1.
[7] Velocity integrated by moving meter slowly from top to bottom and return.
[8] Indeterminate velocity of approach. Crest rounded.
[9] Experiments on 36-inch lock-bar steel pipe.
[10] Rectangular weir with suppressed contractions at both ends. Velocity of approach indeterminate.

Color.—Three chemicals were used in making powerful colors—fluorescein, Congo red, and potassium permanganate. About 1 teaspoonful of fluoreacein (in the form of red powder) mixed in about a pint of water gave sufficient solution for four injections of color in a pipe carrying up to 60 second-feet. The powder mixed readily in cold water. Congo red is used in the same way, but about 10 times as much "red" is recessary to obtain the same intensity of color.

In the use of potassium, a handful of the crystals made about 2 quarts of dense color. Cold water quickly attains a saturate solution if well stirred (from 5 to 8 per cent by weight at ordinary air temperature), and may then be poured off the undissolved crystals, which do not float. About 1 pint of solution will color 50 second-feet of clear water. Some waters break down the color to a muddy brown, which is unsatisfactory.

In making a test the coupling *W* (fig. 2) is opened and the solution poured into the reservoir *S*. After *W* is closed the gun is pumped up like a bicycle tire. Noting the time to a second, the operator opens the cock *V*.

The observer at the outlet notes to the second the first and last appearance of the color. The color is extended by the variation in the velocity throughout the section of the pipe. This extension covers from 8 to 12 per cent of the total time the color spends in the pipe. Comparison with carefully constructed weirs shows that the color method is correct within about 3 per cent. Wherever possible, a comparison between color and current meter was made. To secure comparative results, the time the color spent in the pipe is taken as from the moment of injection to the mean between first and last sight at the outlet. These comparative tests are shown in Table 2.

All watches used in the tests were compared before and after each experiment, corrections being noted for use in computing elapsed time. The watch agreeing best with jewelers' chronometers was used as the standard.

FIELD PROCEDURE.

After the reach of pipe was selected, the manometers attached, and other equipment put in readiness, the method for proceeding with the field test was in general carried out as described in the paragraphs following. Any necessary changes are noted in the text in connection with the description of the individual pipes tested.

The watches used at both ends of the reach were adjusted to agree to the second, and again compared at the end of the observation. Manometers were read at intervals varying from 30 seconds to 2 minutes (depending on the amount of pulsation in the water). If a weir was used to measure the discharge of water, a hook gauge above the weir was read every 2 to 5 minutes, depending on the variation of discharge. If a current-meter measurement was necessary to determine the discharge, it was made either during or immediately following the series of manometer readings, the manometers being watched for appreciable variations of discharge. Where color was used to time the actual velocity of the water it was injected into the pipe at approximately known intervals, say 5 minutes, throughout the time during which the manometers were read. Ordinarily the second gauge was near enough to the outlet of the pipe so that one observer could both read the manometer and watch for the appearance of the color. Sometimes an additional observer was necessary.

OFFICE EQUIPMENT AND METHODS.

Original multiplication, division, and addition were performed on mechanical devices, except in the computation of a few coefficients, where slide rules sufficed. Checking was done by 20-inch slide rules and graphic methods. Estimate diagrams were checked by proving random examples.

Office procedure.—Where water columns were used at both ends of the reach of pipe tested the loss of head in the pipe for the given velocity was the difference in elevation between the top of the mean water column at gauge No. 1 and the top of the mean column of gauge No. 2. Where a mercury manometer was used at one or both of the gauges the equivalent water column for each reading of the mercury column was computed. The mean of the elevations of the tops of the equivalent water columns was accepted as the elevation for that gauge. The loss of head was then computed as before. Standard methods were employed in computing current-meter data or weir discharge. Where color was used in timing the velocity of the water, the time was computed as from the instant of injection to the mean between first sight and last appearance of the color at the outlet.

ELEMENTS OF EXPERIMENTS FOR THE DETERMINATION OF FRICTION LOSSES IN CONCRETE PIPES, FLOWING FULL.

In the following pages two tables are arranged (Tables 3 and 4) which give the elements of all known observations on concrete pipes under pressure. The various series are arranged in ascending sizes of pipe and within one series the observations are arranged in ascending order of velocities.

EXPLANATORY NOTES ON TABLE 3.

Column 1 gives the consecutive numbers of the pipes followed in column 1, Table 4, also in the discussions in the following pages and in the Appendix. The small letter *a* after the numbers refers to discussion in the Appendix. Experiments conducted by this department are discussed in the text while the essential data secured from other sources are abstracted in the Appendix.

Column 2 shows the authority (see also column 2, Table 4) the series number where such was carried, together with the date of the test.

N refers to H. D. Newell, project manager, United States Reclamation Service, Umatilla project, Oregon.

F refers to J. T. Fanning.

B refers to the late Henry Bazin, France.

AB refers to Prof. A. Budau, Austria.

RDJ refers to R. D. Johnston, consulting engineer, Ontario Power Co., Canada.

M refers to F. F. Moore, designing engineer, board of water supply, New York City.

S refers to the writer, Fred. C. Scobey, senior irrigation engineer, in charge of experiments on the flow of water in channels and pipes. In these experiments and computations he was assisted by E. C. Fortier, P. A. Ewing, and Thomas H. McCarthy.

The other columns are considered self-explanatory.

EXPLANATORY NOTES ON TABLE 4.

Column 1 gives the same consecutive numbers as column 1, Table 3. See discussions after "Column 1" on page 20.

Column 10 gives the equation for the pipe, as developed by the tests. This equation is obtained by the "center of gravity" method—the equivalent of the "least squares" method in its simplest form.

The other columns are considered self-explanatory.

TABLE 3.—*Elements of experiments for the determination of friction losses in concrete pipes, running full; with retardation coefficients for various formulas.*

1 Pipe number.	2 Experimenter and year.	3 Observation.	4 Inside diameter. *d*	5 Name and description of pipe.	6 Discharge per second. *Q*	7 Mean velocity, per second. *V*	8 Loss of head, per 1,000 feet. *H*	Coefficients of retardation.					
								9 Scobey. C_s	10 Chezy. C	11 Williams-Hazen. C_w	12 Moritz. C_m	13 Kutten. n	14 Weisbach. f
			ins.		*cu.ft.*	*feet.*	*feet.*						
1	S 19 1915	1	8.0	Pomona, Calif.; jointed cement pipe on continuous down grade, straight; laid about 1885.	1.22	3.51	10.800	0.291	82.6	94.8	0.95	0.0124	.0378
		2			1.24	3.56	10.780	.295	83.8	96.3	.98	.0123	.0357
		3			1.24	3.56	10.820	.295	83.8	96.1	.98	.0123	.0359
2	S 21 1915	2	10.0	Pomona, Calif.; jointed cement pipe on continuous down grade, straight; laid 1888.	1.10	2.02	3.120	.271	79.2	92.9	.96	.0134	.0410
		1			1.12	2.06	3.129	.276	80.8	94.6	.98	.0132	.0396
		3			1.13	2.08	3.334	.270	79.1	92.3	.95	.0134	.0414
		4			1.14	2.10	3.403	.270	79.0	92.3	.95	.0134	.0414
3	S 28 1915	1	10.0	Ontario, Calif.; jointed cement pipe, on continuous grade.	2.59	4.75	15.900	.283	82.6	90.7	.91	.0130	.0378
		2			2.65	4.87	15.850	.290	84.8	93.1	.94	.0126	.0358
4	S 20 1915	1	12.0	Pomona, Calif.; jointed cement pipe, on continuous grade.	1.25	1.60	1.793	.253	76.2	88.4	.91	.0144	.0451
		2			1.25	1.61	1.884	.248	73.2	86.6	.89	.0147	.0468
5	S 26 1915	10	12.0	Pomona, Calif.; jointed cement pipe, straight and approximately level, clear water; laid 1885.	1.18	1.50	1.694	.244	72.8	85.4	.88	.0148	.0485
		9			1.32	1.68	2.120	.244	73.0	84.8	.87	.0148	.0484
		1			1.54	1.95	2.188	.280	83.8	97.3	1.00	.0133	.0367
		2			1.57	2.00	2.497	.268	80.0	92.4	.94	.0138	.0402
		3			1.70	2.17	2.925	.268	80.4	92.0	.94	.0137	.0401
		4			1.80	2.29	3.342	.265	79.2	90.4	.92	.0139	.0411
		5			2.10	2.68	4.423	.270	80.7	90.9	.92	.0137	.0396
		6			2.24	2.85	4.918	.272	81.4	91.3	.92	.0136	.0390
		8			2.71	3.45	7.055	.275	82.1	91.0	.92	.0133	.0382
		7			2.85	3.63	7.649	.278	82.9	91.6	.92	.0135	.0374
6	S 33 1915	1	12.0	Ontario, Calif.; jointed cement pipe, straight; laid 1911.	4.59	5.85	14.820	.322	95.9	103.3	1.03	.0120	.0279
7	S 27 1915	2	12.0	Ontario, Calif.; jointed cement pipe, straight and level; laid 1900.	1.51	1.92	1.405	.343	102.1	121.1	1.25	.0114	.0246
		1			1.57	2.00	1.489	.347	103.6	122.2	1.26	.0112	.0240
8	S 53 1916	1	11.8	Oakdale, Calif.; jointed cement pipe, slight sag in profile.	.98	1.29	1.310	.240	71.8	85.2	.88	.0148	.050
		6			1.18	1.55	1.769	.249	74.4	87.3	.90	.0145	.0465

TABLE 3.—*Elements of experiments for the determination of friction losses in concrete pipes, running full; with retardation coefficients for various formulas*—Continued.

1	2	3	4	5	6	7	8	Coefficients of retardation.					
Pipe number.	Experimenter and year.	Observation.	Inside diameter. d	Name and description of pipe.	Discharge per second. Q	Mean velocity, per second. V	Loss of head, per 1,000 feet. H	9 Scobey. C_s	10 Chezy. C	11 Williams-Hazen. C_w	12 Moritz. C_m	13 Kutten. n	14 Weisbach. f
			ins.		*cu. ft.*	*feet.*	*feet.*						
9	S 24 1915	1	16.0	Pomona, Calif.; jointed cement pipe, straight, on continuous down grade; pipe clean; laid 1883.	1.37	.98	.536	.237	73.1	86.8	.90	.0154	.0478
		2			1.54	1.10	.512	.272	84.6	99.8	1.03	.0132	.0362
		3			1.62	1.16	.570	.272	84.1	99.4	1.02	.0128	.0363
10	S 22 1915	4	16.0	Pomona, Calif.; jointed cement pipe, straight, on continuous down grade; pipe clean; laid 1883.	2.40	1.72	1.001	.304	94.5	108.6	1.11	.0127	.0290
		3			2.47	1.77	1.051	.305	94.6	108.9	1.11	.0127	.0287
		5			2.61	1.87	1.251	.296	91.7	104.7	1.07	.0130	.0306
		2			2.74	1.96	1.315	.302	93.8	106.9	1.09	.0128	.0293
		7			2.82	2.02	1.415	.300	93.1	105.9	1.08	.0129	.0297
		1			2.92	2.09	1.445	.307	95.4	108.2	1.10	.0127	.0283
		6			3.35	2.40	1.964	.303	93.8	105.4	1.07	.0129	.0292
		8			3.41	2.44	2.088	.292	92.4	103.6	1.05	.0129	.0305
11	S 25 1915	1	16.0	Pomona, Calif.; jointed cement pipe, continuous grade.	2.40	1.72	.984	.306	95.0	109.7	1.12	.0127	.0285
12a	N 1 1912	....	16.0	Z_a siphon, Umatilla project, U. S. Reclamation Service, Oreg.	3,74	2.68	3.800	.243	75.0	82.4	.82	.0154	.0453
					4.87	3.49	4.500	.291	90.0	98.0	.98	.0134	.0317
13a	N 2 1912	....	16.0	Z_c siphon, Umatilla project.	3.42	2.45	1.900	.314	98.0	109.5	1.11	.0125	.0271
14	S 58 1916	1	18.0	Edwards line, Oakdale irrigation district, Calif.; straight siphon.	3.48	1.97	1.625	.254	79.8	88.9	.89	.0148	.0405
		2			3.79	2.15	1.930	.254	79.8	88.3	.88	.0150	.0405
15	S 56 1916	1	18.0	Batdorf line, Oakdale irrigation district; straight down grade.	1.70	.96	.309	.284	89.7	106.6	1.11	.0132	.0321
16	S 42 1916	6	19.7	C lateral, Kamloops, B. C., Canada; straight in alignment, wavy in profile, rough joints; laid 1911.	.79	.37	.056	.246	78.1	98.3	1.03	.0133	.0429
		2			1.07	.50	.109	.238	75.8	92.7	.96	.0145	.0459
		3			1.75	.84	.272	.249	79.2	93.6	.95	.0148	.0417
		4			2.35	1.12	.466	.255	81.3	93.7	.95	.0147	.0397
		5			2.63	1.25	.538	.264	84.4	96.8	.98	.0143	.0371
		7			2.93	1.39	.745	.251	79.6	90.6	.91	.0151	.0412
		1			3.12	1.48	.781	.260	82.7	93.9	.94	.0144	.0382
17	S 46 1916	4	19.7	Bishop lateral, Kamloops, B. C., Canada; jointed cement pipe, gentle curve in alignment, on continuous down grade; laid 1911.	1.33	.63	.102	.307	98.6	120.0	1.24	.0124	.0271
		5			1.98	.94	.215	.315	100.0	119.5	1.22	.0121	.0257
		7			3.14	1.49	.557	.310	98.7	113.4	1.14	.0126	.0265
		8			3.70	1.76	.744	.316	100.4	114.2	1.15	.0126	.0255
		6			4.13	1.96	.946	.313	99.5	112.0	1.12	.0127	.0260
18	S 60 1916	5	19.9	Temescal Water Co., Corona, Calif.; inverted siphon of jointed cement pipe reinforced, straight in alignment, double reverse curve in profile; laid 1911.	2.14	.99	.200	.340	108.4	129.4	1.33	.0119	.0220
		4			3.63	1.68	.580	.339	108.1	123.7	1.25	.0118	.0221
		2			4.07	1.88	.700	.346	109.8	125.3	1.26	.0117	.0212
		3			5.13	2.37	1.094	.349	111.3	124.3	1.24	.0116	.0208
		1			5.50	2.54	1.278	.346	110.4	122.5	1.22	.0117	.0212
19	F 1880	1	20.0	Wrought-iron cement-lined pipe, 8,171 feet long according to Schmeer.	2.07	.95	.230	.304	97.4	115.2	1.18	.0124	.0274
		2			3.25	1.49	.440	.345	109.8	127.2	1.29	.0115	.0213
		3			4.20	1.92	.730	.346	110.7	125.1	1.26	.0116	.0212
		4			5.08	2.33	1.040	.351	112.0	125.1	1.25	.0116	.0206
		5			5.67	2.60	1.340	.345	110.1	121.7	1.21	.0117	.0213
		6			6.26	2.87	1.580	.351	111.7	122.8	1.22	.0115	.0205
		7			7.14	3.27	1.990	.356	113.5	123.7	1.23	.0114	.0200
		8			7.50	3.44	2.280	.350	111.7	120.9	1.20	.0116	.0207
		9			8.16	3.74	2.720	.349	111.1	119.6	1.18	.0117	.0209
		10			8.55	3.92	3.000	.348	110.8	118.9	1.17	.0118	.0210
		11			8.73	4.00	3.130	.348	110.7	118.5	1.17	.0118	.0210
		12			8.82	4.04	3.200	.347	110.6	118.3	1.16	.0118	.0210
20	S 57 1916	1	23.7	Clavey siphon, Oakdale irrigation district, Calif.; monolithic, on gentle upgrade; built 1912.	4.77	1.55	.426	.328	106.7	120.9	1.21	.0122	.0226
		2			5.59	1.82	.543	.340	110.7	124.4	1.24	.0119	.0210
		3			6.02	1.95	.522	.373	121.6	136.7	1.36	.0110	.0174

TABLE 3.—*Elements of experiments for the determination of friction losses in concrete pipes, running full; with retardation coefficients for various formulas*—Continued.

1	2	3	4	5	6	7	8	Coefficients of retardation.					
								9	10	11	12	13	14
Pipe number.	Experimenter and year.	Observation.	Inside diameter. d	Name and description of pipe.	Discharge per second. Q	Mean velocity, per second. V	Loss of head, per 1,000 feet. H	Scobey. C_s	Chezy. C	Williams-Hazen. C_w	Moritz. C_m	Kutten. n	Weisbach. f
			ins.		*cu. ft.*	*feet.*	*feet.*						
21	S 47 1916	4	23.5	"A" lateral, Kamloops, B. C., Canada; nominally a 24-inch jointed cement pipe, but obstructed by débris.	1.73	.63	.103	.268	88.4	104.9	.94	.0134	.0338
		2			2.32	.84	.230	.241	78.8	91.2	.81	.0154	.0416
		3			2.65	.97	.296	.243	79.2	91.2	.81	.0155	.0408
		1			3.63	1.32	.495	.257	84.0	94.3	.82	.0150	.0367
22	S 61 1916	1	30.0	Temescal Water Co., Corona, Calif.; inverted siphon, jointed concrete pipe, straight in alignment, sag in profile; laid 1911.	3.25	.67	.094	.261	87.3	102.1	1.03	.0142	.0339
		8			6.38	1.31	.384	.253	84.5	93.6	.92	.0156	.0359
		7			8.23	1.69	.635	.254	84.9	92.0	.90	.0156	.0357
		6			8.28	1.70	.644	.253	85.0	91.8	.90	.0156	.0357
		5			8.38	1.72	.663	.253	84.3	91.5	.90	.0157	.0360
		4			8.38	1.72	.634	.258	86.4	93.7	.92	.0154	.0344
		3			8.52	1.75	.700	.250	83.7	90.4	.88	.0158	.0367
		2			9.11	1.87	.806	.249	83.5	89.5	.87	.0159	.0370
23	S 34 1915	3	30.0	D_1 siphon, Umatilla project, U. S. Reclamation Service, Oreg.; jointed concrete pipe.	5.10	1.04	.150	.320	108.3	123.1	1.23	.0123	.0224
		2			8.74	1.78	.405	.334	112.0	123.3	1.22	.0125	.0206
		1			12.00	2.45	.745	.339	113.0	122.1	1.19	.0125	.0200
24a	N 3 1911	2	30.0	D_1 siphon, same as No. 23 above; tests made when pipe was new and clean.	16.62	3.39	1.050	.395	132.0	140.4	1.37	.0108	.0147
		2			17.04	3.47	1.050	.404	135.0	143.7	1.40	.0106	.0140
		1			17.70	3.61	1.070	.416	140.0	148.0	1.44	.0103	.0132
25a	N 4 1912	1	30.0	R_3 siphon, Umatilla project, U. S. Reclamation Service, Oreg.	13.40	2.73	.860	.351	118.0	126.0	1.23	.0119	.0186
26	S 51 1916	6	30.5	Prosser pressure pipe, Sunnyside project, U. S. Reclamation Service, Wash., jointed concrete pipe reinforced, on continuous down grade, straight and clean; laid 1911.	24.80	4.88	2.230	.386	129.4	133.0	1.28	.0111	.0154
		5			25.60	5.04	2.653	.366	122.6	125.1	1.20	.0116	.0171
		1			26.20	5.15	2.644	.375	125.6	128.1	1.23	.0114	.0163
		4			26.60	5.24	2.869	.365	122.4	124.8	1.19	.0116	.0171
		7			28.80	5.67	3.805	.343	115.2	115.9	1.10	.0123	.0194
		3			28.90	5.68	3.757	.346	116.2	116.9	1.11	.0121	.0191
		2			29.40	5.78	3.727	.354	118.7	119.5	1.14	.0119	.0183
27a	B 1895	1	31.5	Short experimental conduit at Dijon, France, joints perfect, exceeding smoothness indicated by retardation factors.	16.27	3.01	.673	.424	142.4	153.2	1.49	.0104	.0126
		2			18.70	3.46	.925	.416	140.4	148.6	1.44	.0106	.0131
		3			19.78	3.65	1.050	.413	139.5	146.9	1.42	.0106	.0133
		4			22.71	4.20	1.375	.418	139.4	145.7	1.41	.0107	.0132
		5			25.22	4.72	1.725	.416	140.5	145.1	1.38	.0106	.0131
		6			25.93	4.79	1.750	.419	141.7	146.1	1.41	.0106	.0129
		7			26.66	4.92	1.850	.419	141.5	145.7	1.40	.0106	.0129
		8			31.49	5.82	2.575	.420	141.5	144.0	1.38	.0106	.0129
		9			35.66	6.59	3.275	.421	142.3	143.2	1.36	.0106	.0128
28	S 55 1916	4	36.0	Clavey siphon, Oakdale irg. dist., Calif., monolithic pipe, built with wood forms, continuous down grade; built 1912.	12.70	1.80	.438	.290	99.3	106.5	1.04	.0140	.0260
		5			13.40	1.89	.330	.351	120.3	130.6	1.28	.0119	.0178
28b	S 54 1916	1			13.50	1.91	.464	.298	102.2	109.4	1.06	.0137	.0246
		2			13.70	1.94	.581	.270	92.8	98.4	.95	.0150	.0300
		3			13.80	1.95	.587	.271	92.8	98.5	.95	.0150	.0299
29	S 36 1916	1	36.0	Deer Flat Forest pipe line, Boise project, U. S. Reclamation Service, Idaho.	24.60	3.48	1.170	.343	117.6	121.2	1.16	.0123	.0187
29b	S 36b 1917	1		Same reach of same pipe two years later; laid 1912; straight in horizontal alignment.	38.60	5.45	2.072	.403	138.3	139.4	1.33	.0110	.0134
		2			50.00	7.12	3.612	.399	136.5	134.8	1.26	.0110	.0137
		6			52.60	7.44	3.509	.423	145.2	143.1	1.35	.0105	.0123
		4			58.10	8.22	5.328	.379	130.1	126.2	1.18	.0115	.0152
		5			61.30	8.66	5.321	.400	137.2	133.2	1.25	.0108	.0136
		3			64.10	9.06	5.30	.419	143.8	139.6	1.31	.0105	.0124
30	S 49 1916	5	42.0	Victoria aqueduct, B. C., Canada; siphon No. 1, jointed concrete pipe, reinforced; much curvature, extremely smooth interior, joints included; conditions unusually favorable for experimentation; laid 1914.	9.68	1.01	.076	.354	123.9	139.6	1.33	.0112	.0168
		6			11.45	1.19	.097	.370	129.2	144.2	1.37	.0113	.0155
		7			13.56	1.41	.136	.370	129.2	142.4	1.34	.0111	.0154
		8			15.97	1.66	.179	.380	132.7	144.5	1.31	.0110	.0146
		1			16.84	1.75	.212	.368	128.5	139.1	1.34	.0114	.0156
		9			18.86	1.96	.253	.377	131.7	141.6	1.36	.0112	.0148
		2			21.55	2.24	.316	.385	134.9	143.5	1.32	.0110	.0142
		3			24.34	2.53	.415	.380	132.8	139.9	1.28	.0112	.0146
		4			27.99	2.91	.530	.387	135.1	141.0	1.26	.0111	.0141

TABLE 3.—*Elements of experiments for the determination of friction losses in concrete pipes, running full; with retardation coefficients for various formulas*—Continued.

1	2	3	4	5	6	7	8	Coefficients of retardation.					
Pipe number.	Experimenter and year.	Observation.	Inside diameter. d	Name and description of pipe.	Discharge per second. Q	Mean velocity, per second. V	Loss of head, per 1,000 feet. H	9 Scobey. C_s	10 Chezy. C	11 Williams-Hazen. C_w	12 Moritz. C_m	13 Kutten. n	14 Weisbach. f
			ins.		*cu. ft.*	*feet.*	*feet.*						
31	S 48 1916	1	42.0	Victoria aqueduct, siphon No. 5 (see No. 30 above); series not so trustworthy as No. 30; reach much shorter; ignore observation 1.	9.50	.99	.042	.466	163.0	188.2	1.88	.0084	.0097
		5			18.50	1.92	.214	.402	140.4	151.8	1.48	.0105	.0131
		4			20.90	2.17	.275	.400	139.9	149.7	1.45	.0106	.0132
		3			21.60	2.24	.310	.389	136.2	145.1	1.40	.0109	.0139
		2			24.90	2.59	.429	.382	133.7	140.6	1.35	.0112	.0145
32	S 35 1915	3	46.0	R_1 siphon, Umatilla project, U. S Reclamation Service, Oreg.; jointed concrete.	16.20	1.41	.096	.416	147.0	162.4	1.59	.0099	.0119
		2			25.10	2.18	.252	.397	139.7	149.0	1.44	.0108	.0131
		1			36.50	3.17	.544	.393	139.0	143.1	1.36	.0109	.0134
33a	N 5 1911 1912	2	46.0	R_1 siphon, same as No. 32 above; tests made when pipe was new.	45.90	3.98	780	.412	145.0	147.9	1.40	.0106	.0122
		1			46.20	4.00	.870	.392	138.0	140.2	1.33	.0111	.0134
		3			48.20	4.17	1.080	.375	130.0	129.9	1.23	.0117	.0153
		4			48.60	4.21	1.020	.381	135.0	135.3	1.28	.0113	.0142
33b	S 60	1	48.0	Churn Creek siphon, Anderson-Cottonwood irg. dist., Calif.	47.40	3.77	1.145	.313	111.0	111.0	1.04	.0137	.0208
34	S 52 1916	1	54.0	Mabton pressure pipe, Sunnyside project, Wash.	80.80	4.99	1.249	.368	132.7	129.4	1.20	.0118	.0146
35	S 40 1915	6	63.5	Simms Creek siphon, Sun River project, U. S. Reclamation Service, Mont.; monolithic pipe, straight in alignment; built 1907–8	66.40	3.02	.289	.420	154.1	156.7	1.47	.0104	.0108
		1			78.00	3.55	.377	.432	159.2	159.6	1.49	.0101	.0102
		5			86.60	3.94	.459	.434	160.2	159.2	1.49	.0101	.0101
		4			123.30	5.61	1.194	.384	141.0	135.3	1.24	.0114	.0129
		2			130.60	5.94	1.286	.391	143.8	137.7	1.26	.0112	.0124
		3			140.40	6.39	1.581	.380	133.7	132.4	1.21	.0116	.0132
36a	AB 1913	1	86.6	Perlmoos cement works, Austria; gentle curves in alignment, on continuous down grade; original data do not describe pipe manufacture or surface.	58.40	1.43	.066	.344	130.9	136.1	1.28	.0123	.0148
		2			90.50	2.21	.155	.345	132.4	131.9	1.23	.0124	.0147
		3			113.90	2.78	.181	.402	153.7	152.6	1.42	.0108	.0109
		4			131.90	3.22	.312	.355	136.0	131.8	1.21	.0122	.0140
		5			152.10	3.68	.402	.358	137.0	131.4	1.21	.0122	.0138
		6			166.70	4.08	.496	.356	136.3	129.7	1.18	.0122	.0139
37a	M 1		110.0	Catskill aqueduct siphons..	542.00	8.21	1.150	.405	160.0	142.7	1.26	.0109	.0101
38	S 59 1916	4	120.0	Whitney siphon, Los Angeles aqueduct, Calif., monolithic pipe, cement mortar coat.	223.00	2.84	.191	.326	130.4	123.4	1.12	.0133	.0152
		3			236.00	3.01	.211	.328	131.2	123.6	1.12	.0132	.0151
		2			238.00	3.03	.284	.285	113.5	106.1	.96	.0153	.0199
39a	M 2 1915	5	174.0	Rondout tunnel, Catskill aqueduct, N. Y.; monolithic tunnel lining made with oiled steel forms, observations made on consecutive days.	260.90	1.58	.031	.357	150.0	144.8	1.31	.0124	.0116
		4			345.00	2.09	.051	.368	155.0	146.4	1.32	.0119	.0109
		6			469.00	2.84	.112	.338	140.5	130.1	1.18	.0131	.0130
		3			472.30	2.86	.089	.381	159.0	148.4	1.32	.0116	.0102
		2			620.90	3.76	.155	.380	159.0	144.5	1.28	.0116	.0102
		7			624.20	3.78	.211	.327	137.0	123.0	1.08	.0134	.0138
		8			756.30	4.58	.319	.323	135.0	119.2	1.04	.0136	.0142
		1			771.20	4.67	.253	.369	154.0	137.8	1.21	.0119	.0108
40a	M 3 1915	6	174.0	Wallkill tunnel, Catskill aqueduct, N. Y., monolithic tunnel lining made with oiled steel forms; observations made on same day that identical discharge held in pipe No. 39 above, which see.	260.90	1.58	.033	.346	143.5	140.1	1.27	.0129	.0123
		5			345.00	2.09	.051	.368	154.0	146.4	1.35	.0120	.0109
		7			469.00	2.84	.107	.345	144.0	133.4	1.19	.0128	.0124
		4			472.30	2.86	.092	.374	157.0	145.7	1.30	.0118	.0105
		3			620.90	3.76	.192	.341	143.0	128.7	1.14	.0129	.0127
		8			624.20	3.78	.170	.365	152.0	138.2	1.22	.0121	.0111
		1			736.50	4.46	.249	.356	149.0	132.7	1.17	.0124	.0117
		9			756.30	4.58	.261	.357	149.0	132.9	1.17	.0123	.0116
		2			771.20	4.67	.269	.358	149.5	133.2	1.17	.0123	.0115
41a	RDJ 1913 1914		216.0	Conduit No. 2, Ontario Power Co., Canada. See p. 84 for explanation of items in this table. They do not represent data on actual individual runs, but are from velocity-friction curve.	1,018.00	4.00	.108	.424	181.0	163.0	1.436	.0104	.0098
					2,036.00	8.00	.448	.416	178.0	152.0	1.304	.0105	.0102
					3,045.00	12.00	.990	.420	180.0	148.0	1.256	.0104	.0100
					4,063.00	16.00	1.701	.427	183.0	148.0	1.242	.0102	.0097
					5,091.00	20.00	2.397	.450	193.0	153.0	1.286	.0097	.0087

TABLE 4.—*Summary of series of experiments upon concrete pipes running full, including individual pipe equations. To be considered as supplementary to table 3.*

1	2	3	4	5	6	7	8	9	10
Pipe No.	Experimenter.	Age of pipe.	Maximum pressure head.	Diameter of bore.	Diameter of bore.	Length of reach tested.	Area of bore.	Range of velocities.	Individual pipe equations.
		Years.	*Feet.*	*Inches.*	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	
1	Scobey	30	7	8.0	0.67	1,310.9	0.349	3.51- 3.56	
2	do	27	5	10.0	.83	1,107.0	.545	2.02- 2.10	
3	do	10	5	10.0	.83	936.2	.545	4.75- 4.87	
4	do	30	4	12.0	1.00	621.1	.785	1.60- 1.61	
5	do	30	4	12.0	1.00	182.8	.785	1.50- 3.63	$H=0.7406 V^{1.813}$.
6	do	4	8	12.0	1.00	217.8	.785	5.85	
7	do	15	5	12.0	1.00	869.4	.785	1.92- 2.00	
8	do	......	8	11.8	.98	1,850.8	.762	1.29- 1.55	
9	do	32	4	16.0	1.33	785.0	1.396	0.98- 1.16	
10	do	32	2	16.0	1.33	853.8	1.396	1.72- 2.44	$H=0.3464 V^{1.985}$.
11	do	32	2	16.0	1.33	1,897.6	1.396	1.72	
12a	Newell	2	18	16.0	1.33	....	1.396	2.68- 3.49	
13a	do	2	18	16.0	1.33	....	1.396	2.45	
14	Scobey	......	11	18.0	1.50	1,271.5	1.767	1.97- 2.15	
15	do	......	3	18.0	1.49	559.3	1.703	.96	
16	do	3	10	19.7	1.64	1,116.0	2.106	.37- 1.48	$H=0.3727 V^{1.863}$.
17	do	4	6	19.7	1.64	771.8	2.106	.63- 1.96	$H=0.2484 V^{1.981}$.
18	do	4	80	20.0	1.66	2,163.2	2.166	.99- 2.54	$H=0.2116 V^{1.900}$.
19	Fanning	......	......	20.0	1.67	....	....	.95- 4.04	$H=0.2186 V^{1.903}$.
20	Scobey	3	40	23.8	1.98	1,046.2	3.079	1.55- 1.95	
21	do	5	7	24.0	2.00	2,306.5	3.142	.63- 1.32	$H=0.2978 V^{2.078}$.
22	do	4	8	29.9	2.49	880.0	4.870	.67- 1.87	$H=0.2272 V^{2.054}$.
23	do	5	45	30.0	2.50	5,026.4	4.909	1.04- 2.45	$H=0.1394 V^{1.862}$.
24a	Newell	1	45	30.0	2.50	....	....	3.39- 3.61	
25a	do	4	25	30.0	2.50	....	....	2.73	
26	Scobey	4	50	30.5	2.54	2,276.3	5.075	4.88- 5.78	$H=0.01478 V^{3.180}$.
27a	Bazin	New.	......	31.5	2.62	131.0	5.413	3.00- 6.59	$H=0.0811 V^{1.971}$.
28	Scobey	3	30	36.0	3.00	1,933.6	7.070	1.80- 1.89	
28b	do	3	30	36.0	3.00	1,266.0	7.070	1.91- 1.95	
29	do	3	70	36.0	3.00	7,282.0	7.070	3.48	
29b	do	5	70	36.0	3.00	7,285.0	7.070	5.45- 9.06	$H=0.0497 V^{2.166}$.
30	do	1	65	42.0	3.50	1,336.1	9.620	1.01- 2.91	$H=0.0711 V^{1.889}$.
31	do	1	......	42.0	3.50	378.0	9.620	.99- 2.59	$H=0.0432 V^{2.423}$.
32	do	5	110	46.0	3.82	9,774.0	11.520	1.41- 3.17	$H=0.0456 V^{2.166}$.
33a	Newell	1-2	110	46.0	3.83	9,831.0	....	3.98- 4.21	
33b	Scobey	4	25	48.0	4.00	4,242.3	12.570	3.77	
34	do	7	60	54.0	4.54	2,167.4	16.188	4.99	
35	do	7	40	63.5	5.29	1,138.0	21.979	3.02- 6.39	$H=0.0787 V^{2.400}$.
36a	Budau	New.	38	86.6	7.22	4,200.0	40.919	1.43- 4.08	$H=0.03345 V^{1.912}$.
37a	Moore	......	Var.	110.0	......	....	....	....	
38	Scobey	3	65	120.0	10.00	857.4	78.540	2.84- 3.03	
39a	Moore	New.	710	174.0	14.50	8,419.0	165.130	1.58- 4.67	$H=0.01113 V^{2.109}$.
40a	do	New.	710	174.0	14.50	13,941.0	165.130	1.58- 4.67	$H=0.01203 V^{2.029}$.
41a	Johnston	4	......	......	18.00	6,466.0	254.560	4.00-20.00	$H=0.0082 V^{1.93}$.

DESCRIPTION OF PIPES.

The descriptions in the following pages are to be taken as supplementing Tables 3 and 4. The methods of determining the hydraulic elements necessary for each observation are described. The descriptions of pipes upon which previous experimenters have made observations are given in the appendix.

No. 1, Experiment S-19.—8-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—This length of pipe line, between boxes 86 and 89 on Reservoir Street, near Pomona, is straight in both vertical and horizontal alignment. From an examination of its inlet it is probably safe to say that joints in this, as in so many other of the older pipes, were not as carefully made as they are at present.

Water columns were used for both gauges, No. 1 being attached to a brass piezometer tube of type A, thrust 5.8 feet down the 8-inch pipe from the 24-inch riser pipe similar to the one in Plate III, figure

2. Water entered the riser from an 8-inch pipe directly opposite the pipe line tested. In the reach, 1,310.9 feet long between gauges there are 3 smaller riser pipes. Gauge No. 2 is attached to a piezometer identical in construction with No. 1, held in a position 3 feet above the outlet of the pipe.

The water for this system is clear, being pumped from wells in the "Palomares Cienaga."[1] So far as the writer was able to determine from the inlets and outlets of pipes in this system, there was no deposit of either vegetable or mineral nature.

The nominal diameter of the pipe was used in computing the area and quantity. As the velocity was found by the direct method, any difference between the true and the nominal area does not affect the velocity. If methods of construction used in the eighties were the same as to-day, it is possible the pipe is undersized, as that appears true of most present-day pipes.

As it was not practicable materially to vary the discharge in the pipe the three observations were taken at about the same velocity, although extended over two days. The velocity for any one run was taken as the mean of three batches of fluorescein, injected with a "color gun" into the intake of the pipe line at the foot of the riser pipe at gauge 1, and observed in the low riser pipe at gauge 2. Water was immediately withdrawn from the pool so that there was no uncertainty due to color lagging at the outlet.

The capacity of this pipe is slightly greater than that of most pipes built in the eighties, but it appears to be absolutely clean, though with rough joints. The value of C_s is about 0.292.

No. 2, Experiment S-21.—10-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—This pipe line, between boxes 364 and 366, was laid in 1888. The gauges and piezometer connections were identical with those used and described under No. 1. Piezometer No. 1 was placed 6 feet down the pipe line from the riser pipe forming delivery box No. 364 while piezometer No. 2 was thrust 1.7 feet into the pipe against the current, from box No. 366. A reach of line 1,107 feet long, straight in alignment and profile, was thus tested. Three batches of fluorescein were timed for each observation, the mean time being accepted in computing velocity of the water. The nominal diameter was accepted in computing areas and quantity of water (see No. 1, p. 25). So far as is known, this line had never been disturbed since it was laid. There is probably little or no sediment in the line, as the water is clear. The capacity of this line is slightly more than usual in this vicinity, the value of C_s being about 0.272.

No. 3, Experiment S-28.—10-inch jointed cement pipe, San Antonio Water Co., Ontario, Calif.—This experiment was conducted on a straight reach of pipe laid in 1905 diagonally through an orange grove. In the 10 years since then it has not been necessary to dig up the pipe for root troubles. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). Both gauges consisted of piezometer tubes of type A leading to water columns in glass. The orifices for gauge 1 were located 12.8 feet down the pipe from its intake in a masonry division box, while those for gauge 2 were 936.2 feet farther downstream, and 2 feet above the

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[1] The Use of Underground Water for Irrigation at Pomona, Calif., by C. E. Tait, O. E. S. Bul. 236, U. S. D. A., p. 35.

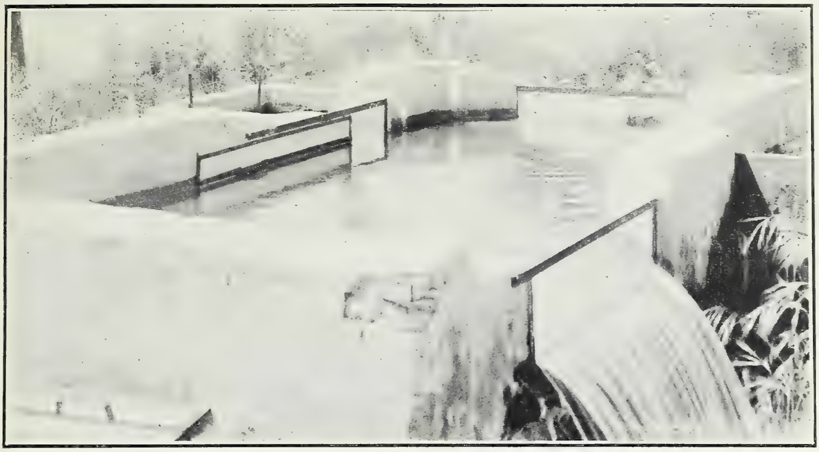

FIG. I.—DIVISION BOX OVER PIPE LINE SYSTEM IN CALIFORNIA.

Where color is used to determine velocity in pipe entering such a box, the last trace of color can not be determined directly, as whole pool is then tinted. See page 14.

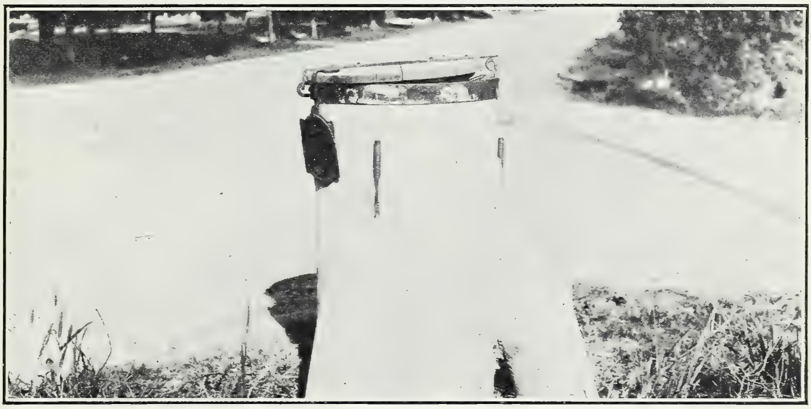

FIG. 2.—DIVISION STAND ON PIPE SYSTEM IN CALIFORNIA.

Piezometer tube is thrust into pipe line and pressure head siphoned over edge of stand to glass gauge tube. Note small iron pipes, in order to allow escape of air near inlet to pipes conveying water from the stand. See page 53.

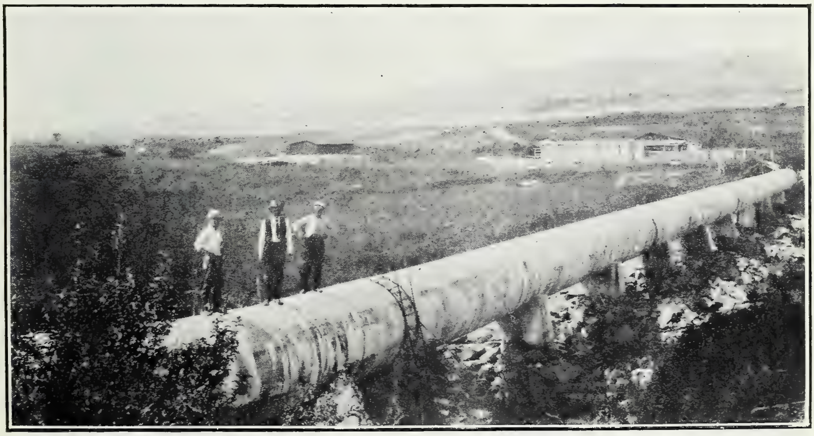

FIG. 3.—SIMMS CREEK SIPHON, SUN RIVER PROJECT, U. S. RECLAMATION SERVICE, MONTANA. (PIPE NO. 35.)

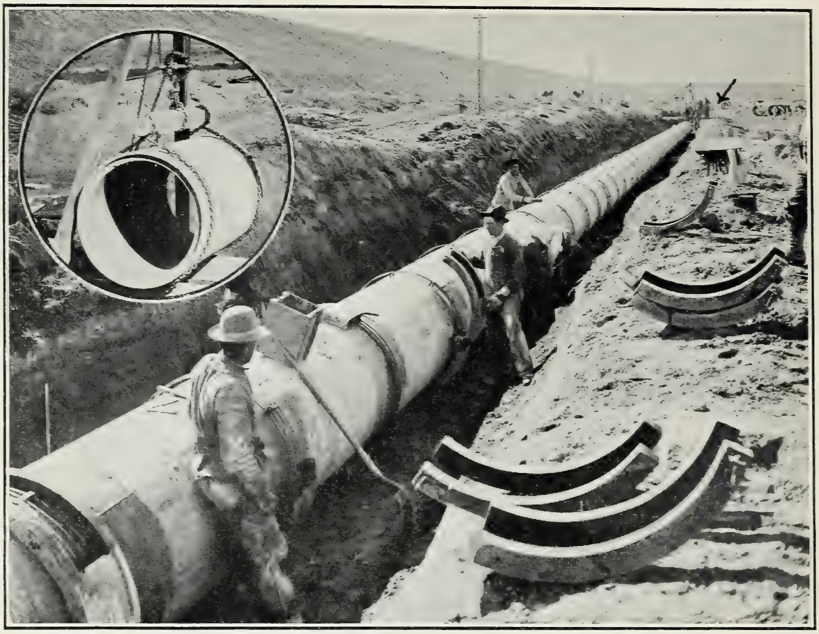

FIG. 1.—DEER FLAT FOREST PIPE LINE, BOISE PROJECT, U. S. RECLAMATION SERVICE, IDAHO. (PIPE NO. 29.)

Note reinforcement and forms for collar joints. Intake at end of trench in distance. Inset shows pipe unit over trench.

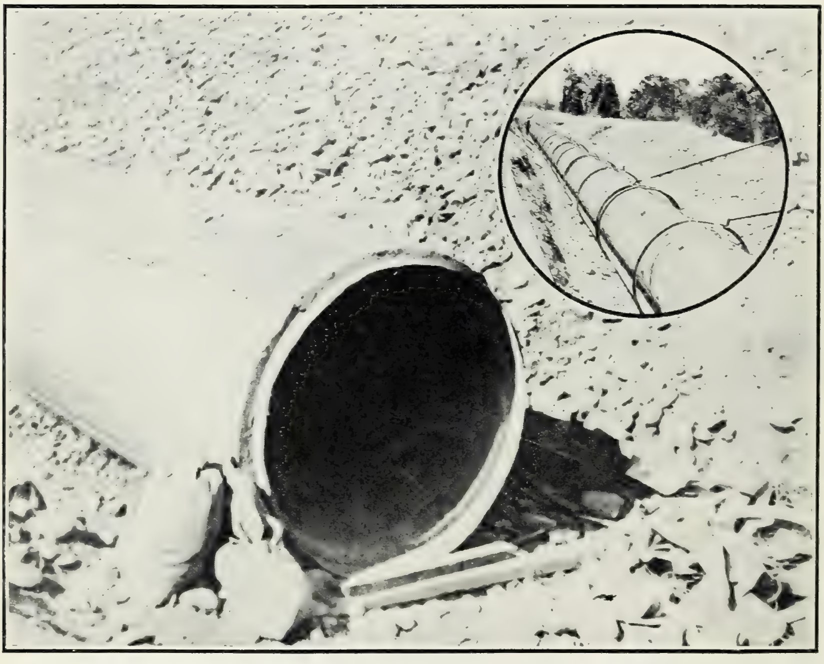

FIG. 2.—CHURN CREEK SIPHON, ANDERSON-COTTONWOOD IRRIGATION DISTRICT, CALIFORNIA. (PIPE NO. 33b.)

Longitudinal ridges on interior surface similar to those shown on exterior; due to cracks between form boards. Inset shows inside forms set for monolithic pouring.

outlet of the pipe to another division box. The mean velocity of four batches of color was taken as the velocity of the water for each of the two runs of water. Unfortunately it was not practicable to vary to a marked degree the quantity flowing in the pipe. The high velocities in this pipe and the clear water probably assure a pipe free from sediment. This is borne out by the value of $C_s = 0.287$, the increase in this coefficient marking the improvement in the joints.

No. 4, Experiment S-20.—12-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—After passing through the reach of pipe described as No. 1, water may be delivered to the consumer or enter this reach of 12-inch pipe between boxes 89 and 90, on Franklin Street, Pomona.

Water columns were used for both gauges, which were attached to peizometer tubes of type A, No. 1 being placed 10 feet down the line from the inlet and No. 2 being thrust 2 feet up from the outlet. Velocities were computed from the mean time of three batches of fluorescein, poured into the inlet and noted at the outlet. The color dragged out from 10 to 14 per cent of the total time spent in the pipe, indicating a very rough interior.

Little is known of the condition of this pipe, as it was laid about 1885. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). So far as could be ascertained at the inlet and outlet, the line is free from silt. It has never been necessary to dig up the pipe on account of root troubles, although it is very close to a row of trees. The line is buried about 3 feet, is straight, and nearly level. The water is clear at all seasons of the year. The poor joints are indicated by the coefficient 0.250.

No. 5, Experiment S–26.—12-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—Water for this irrigation system is secured from wells located on a gravel and boulder subterranean reservoir, locally termed a "cienaga." Dikes of impervious material cut off the flow in the gravel, forming a natural reservoir. The water from several wells is piped to one circular collecting chamber, while two pipes from this chamber permit the water to be conveyed to either one or both of two portions of the main pumping plant. One of these latter pipes 193 feet in length was tested. With a constant draft, the discharge of either pipe could be varied by head gates at the upper end of each pipe. Thus it was possible to vary the velocity in the pipe under test from 1.50 to 3.63 feet per second. For each observation the mean velocity of from 5 to 10 weighted floats was accepted as the velocity of the water between the intake chamber and a vertical riser pipe. These floats were made as follows: Small wire nails were thrust into an orange until it was of the same specific gravity as the water, showing little tendency to either float or sink. The pipe tested was under about a 4-foot head throughout its length, so that there was no tendency for the oranges to become heavier during passage through the pipe due to increasing water pressure, as is the case when a float of this nature is tried on a siphon pipe where the maximum head is much greater than the depth of the water in which the orange is tested while nails are being inserted. Of course, if a float in a pipe is too light it drags against the top of the pipe and if too heavy it drags along the bottom, in either case being impelled by velocities slower than the mean velocity in the pipe. It was thought that the round form of the orange would cause it to roll easily whenever the current carried it against the periphery of the pipe, rather than lurch back into the current, as would a more irregular float. The maximum variation in time of any one float from the mean time of all of the floats was about 5 per cent. In connection with tests on pipe No. 7, floats of this type agreed with the velocity as determined by a well-made, contracted, rectangular weir 2.005 feet long within 1.2 per cent. At the intake a hook gauge in a stilling box determined the pressure head at the piezometer tube (type A) located 10 feet down the pipe. At the outlet a similar tube was held 1.6 feet in the pipe against the current. The pressure head at this tube was read in a water column beside the riser pipe. This pipe was laid in 1885 and appeared to be perfectly clean. As it is in an open field there is no chance of roots choking the interior. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). The value of C_s varies with the velocity, from 0.244 to 0.278.

No. 6, Experiment S-33.—12-inch jointed cement pipe, San Antonio Water Co., Ontario, Calif.—This pipe lies between two division boxes and extends under a wash located north of Twenty-fourth Street and west of Euclid Avenue near Ontario. Though too short for experimental purposes, if low velocities are to be considered the 227.5 feet between the boxes is ample when the commercial velocity, nearly 6 feet per second, is obtainable. Water columns were used at both gauges. These were attached to piezometer tubes of type A, No. 1 being placed 7.7 feet in the pipe from the inlet chamber and No. 2 being held 2 feet up in the pipe from the outlet chamber. Velocities were computed from the mean time of four batches of fluorescein, timed from the instant of injection at the inlet to the mean between the first and last appearance at the outlet. Just four seconds (by test) was required for a given body of water to pass through the length of hose under a head of 1 foot. This head is of course the difference in elevation between the surface of the water in the chamber and the outlet of the hose. The nominal diameter was accepted in computing area and quantity (see No. 1, p. 25).

This pipe line was laid in 1911. The units are 2 feet long, made with a dry mix tamped into steel molds and afterwards washed on the inside with cement grout. The line is buried 2 or 3 feet and is straight but for a slight sag under the wash. The value of C_s, 0.322, shows the marked improvement in joints in pipe line laid in recent years.

No. 7, Experiment S-27.—12-inch jointed cement pipe, San Antonio Water Co., Ontario, Calif.—A straight, level reach of lateral pipe on Fourth Street, Ontario, was chosen for this test. Water enters the pipe over a division wall in a sand box and leaves it at the bottom of a similar sand box on the corner of Fourth and Euclid Streets. Piezometer tubes of type A were used at both ends of a reach 869.4 feet long. The pressure head at gauge 1 was read in a water column beside the sand box, while that at the outlet piezometer was read by a hook gauge in a stilling box attached to the piezometer by pressure tubing. Piezometer No. 1 was 4.7 feet from the inlet and No. 2 was 1.9 feet from the outlet. The mean time of six weighed orange floats like those described under No. 5 was used in computing the velocity of the water in the pipe. Accepting the mean area of samples of pipe of this size in the pipe yards of this company and comparing the velocity as above determined with that found by dividing the discharge by this mean area, shows agreement with the mean of the two measurements of 1.2 per cent. The discharge was measured over a 2-foot contracted, rectangular sheet-iron-crested weir, with end contractions greater than $2h$, bottom contraction about 5 feet, and h less than one-sixth of the length of the weir. The water welled up from the bottom, so that velocity of approach was negligible. This pipe was laid in 1900. A comparison of the zone occupied by points representing this pipe with that of pipes 1, 2, 3, 4, 5, 9, 10, and 11 shows the progress made since the early eighties in the matter of smoother pipe interiors and better joints, the value of C_s for this line being 0.345, an exceptionally high value for "dry-mix" pipe.

No. 8, Experiment S–53.—12-inch jointed concrete pipe, J. W. Crane private lateral near Oakdale, Calif.—From one of the laterals of the Oakdale irrigation district water is conveyed down a gentle hillside to the fields by means of a concrete pipe line. A reach 1,850.8 feet long was chosen for test, between the intake and an outlet hydrant in a riser pipe. The reach has one gentle curve in horizontal alignment and a slight sag near the far end. Water columns were used for both gauges, which were attached to piezometer tubes of type A. No. 1 was placed 4.4 feet down the pipe from the intake chamber, while No. 2 was thrust up into the line 6.4 feet from the riser pipe. Gauge No. 2 was a glass tube, while gauge No. 1 was a hook gauge noting the surface of the water in a stilling box. Thus a correction for capillarity was necessary in computing the elevation of the water column for gauge No. 2. Between the inlet and the second gauge are 21 riser pipes each 1 foot high, capped with a hydrant. The interior of the pipe was examined through several of these hydrants and found to be clean of silt, but with rough joints between the 2-foot units of pipe. The line was constructed by a man rather new in the business and the value of smooth joints was not appreciated as it now is in southern California. This is shown by the fact that C_s is but 0.245. Measurements of the pipe interior through these hydrants showed the average diameter to be 0.985 foot or that the area of the pipe was about 97 per cent of the area of a 1-foot pipe. The measured area was used in computations. Velocities were determined by injecting saturate solution of potassium permanganate into the upper end of the line and observing the first and last appearance of the color at an open hydrant 55 feet beyond the hydrant at gauge No. 2. A few small leaks were evidenced by moist ground, but the use of color in determining velocities automatically takes care of any correction for such leaks, the velocity of the color dropping in proper proportion and at the proper time as each leak is reached.

No. 9, Experiment S–24.—16-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—A second reach of the same main pipe line discussed as No. 10 was tested between boxes 91 and 92. This part of the line runs diagonally under an orange grove, but, although it was laid in 1883, roots have never interfered with the flow of water. It is straight, on a gentle down grade, and buried about 2 feet. There was one riser pipe in the reach tested. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). Except for run 1, both gauges consisted of hook gauges in stilling boxes, No. 1 being attached by pressure tubing to a brass piezometer of type A, located 16.2 feet down the pipe line, while gauge No. 2 records the elevation of the water in a delivery box 785.1 feet from gauge No. 1. For run 1, readings were taken by a plumb bob attached to a steel tape and allowed to scratch the water surface in the riser pipe at gauge 1. Water entered and left this riser pipe in the same straight line, so insuring that there was practically no loss of head within the riser (see No. 1). Examination of the inlet and outlet of this line indicated that there was no deposit upon the surface of the pipe. The mean time of two batches of fluorescein was accepted in computing the mean velocity of the water. The capacity of this line is about as indicated in Table 6.

No. 10, Experiment S-22.—16-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—This reach of pipe, laid in 1883, comprises part of a main trunk line conveying water from the pumping plant to minor pipe laterals. A straight section between boxes 36 B and 37 was chosen for test. Box 36 B is a single piece of riser pipe set directly over a rough hole in the trunk line about 8 inches square. The surface of the water in the riser was quiet so that hook gauge readings could be taken directly, no stilling box being necessary. Gauge and piezometer No. 2 were like those described under No. 9, the piezometer tube being held 3.2 feet up the pipe line against the current. The water was clear and no sediment in the pipe was seen through the hole at box 37 B mentioned above. The pipe is on an even down grade, at no place under more than 3 feet of head. From three to five batches of fluorescein were timed for each run of water and the mean time was accepted in computing the mean velocity of the water for that particular observation. In order to secure runs at varied discharges it was necessary to return to this pipe from day to day and take an observation at the discharge then carried. The regimen of flow for each run was thus fully established. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). The line is well laid, in comparison with other 1883 pipes, the value of C_s being about 0.304.

No. 11, Experiment S-25.—16-inch jointed cement pipe, Irrigation Co. of Pomona, Calif.—Immediately upstream from the reach of pipe discussed as No. 10, a section of the same 16-inch pipe, 1,897.6 feet long, between a masonry division box and delivery box 36 B was available for test. It was practicable to secure but one observation on this line, however, for the reason that the pipe was full at the intake end only when the maxium amount of water then needed was flowing. The nominal diameter was accepted in computing area and quantity of water (see No. 1, p. 25). A hook gauge in a stilling box was used at the upper end. The stilling box was attached by pressure tubing to a piezometer tube of type A, thrust 8.4 feet down the pipe line. Gauge No. 2 of this reach was gauge No. 1 of reach No. 10 (for description of which see this page). This pipe is laid on a gentle down grade with one S curve in horizontal alignment as it is carried around a residence. At no point is it under more than 3 or 4 feet of head. Examination at inlet and outlet showed this pipe to be free from sand or deposits. The water is clear at all times, being pumped from a subterranean bowlder reservoir (cienaga). A glance at Plate VI shows that the loss of head in this reach is practically identical with that in the adjoining reach (No. 10), upon which several observations were possible. The value of C_s is 0.306.

No. 14, Experiment S–58.—18-inch jointed cement pipe, Edwards private line, in Oakdale irrigation district, Calif.—Water for irrigation is conveyed across a depression from a main lateral of the Oakdale district to a knoll, in a plain cement-pipe inverted siphon 1,283.6 feet long, subject to a maximum head of about 11 feet. The pipe was made on the ground by the owner of the farm, aided by a man with a small amount of practical knowledge of pipe making. There is one 12-inch standpipe and one 12-inch valve near the low point. As both inlet and outlet of this line were submerged and it was not possible to vary the discharge to secure much variation in the velocity, practically all of the line was used in the experiments. Water columns attached to piezometer tubes of type A were used at both ends of the reach. Piezometer No. 1 was set 5.1 feet down the pipe from the intake chamber and No. 2 was thrust upstream from the outlet chamber a distance of 7 feet. Velocities were determined by the use of a saturate solution of potassium permanganate, injected at the inlet and observed at the outlet of the siphon. The nominal size of the pipe was accepted, as it was not feasible to secure measurements of the pipe itself.

From the fact that the value of C_s is very low, about 0.254, the writer would judge that sediment has obstructed this pipe more or less.

No. 15, Experiment S-56.—18-inch jointed cement pipe, Batdorf line, Oakdale irrigation district, Calif.—Irrigation water is conveyed from one open-channel lateral to another down a gentle hillside and across about 600 feet of level field by means of an 18-inch cement pipe made of jointed units each 2 feet long. There are four vertical standpipes rising above the hydraulic gradient on the total length of about 2,500 feet. Examination of the pipe at these standpipes showed that the line was full of water only through about the last 700 feet. A reach was chosen from a standpipe to the outlet, a length of 582.3 feet. A tube of type A was dropped down the standpipe and carried by the current 2 feet down the pipe. A similar tube was thrust 21 feet up the pipe from the outlet. Gauge No. 1 was a water column in a glass tube, while piezometer tube No. 2 was connected with a stilling box and the surface of the water determined by a hook gauge. The velocity was determined with a solution of potassium permanganate, timed from the standpipe to the outlet. As it would have been a difficult matter to determine the moment of injection down a standpipe 10 or 15 feet high above the pipe line, the solution was placed in a corked bottle held in a sack made from a piece of fly screen which was wired to the end of a long one-eighth-inch iron pipe. The bottle was thrust down the standpipe and smashed against the bottom of the pipe line, thus releasing the color but retaining the glass within the screen. The line under the standpipe was clean of deposit, as nearly as could be determined by feeling with the iron pipe rod, and from the appearance of the outlet. The value of the friction factor would indicate this to be true. The pipe is straight, buried about 3 feet, appeared from inspection of the outlet to have good joints, and was probably in good condition. The value of C_s is 0.284.

No. 16, Experiment S-42.—20-inch jointed concrete pipe, lateral C, British Columbia Fruitlands Co., Kamloops, British Columbia, Canada.—The main canal on this project is concrete lined, skirting the foot of a low range of hills and delivering water to the intakes of the cement-pipe laterals. This delivery consists of an iron head gate at the canal end of a tube through the canal bank. Water is discharged through the tube into a rectangular sand and pressure regulating box 4 by 5 feet and 4½ feet deep.[1] The water leaves this concrete box over a weir crest and falls to the intake chamber of the pipe line. At the head of lateral C the weir is of heavy sheet iron, rectangular in form, 3.02 feet long, without immediate end contractions, but with a bottom contraction of 39 inches. In addition to the flow over the weir there was a small constant leak through the drain gate near the bottom of the weir wall. This leak was measured and added to the flow over the weir for any particular run of water. A reach of pipe 1,116 feet long was chosen for test between a standpipe located 99 feet from the intake and a division box. The pipe was straight in horizontal alignment but wavy in the vertical plane, having two summits, each protected by standpipes. At low points of the profile the pipe is provided with blow-off valves. The maximum pressure on this pipe is due to a head of about 15 feet at the low points. Water columns attached to piezometer tubes of type A were used for both gauges. Tube No. 1 was dropped down a standpipe and allowed to be carried 3 feet down the pipeline. Tube No. 2 was thrust 1 foot upstream into the pipe line from the wall of the division box. The internal size of the pipe was determined by measuring two diameters on each of 10 pipe units remaining from the original construction. The mean area of these units was accepted as the internal area of the pipe line. Each unit was 2 feet in length. They were made by the "dry mixture" process and afterwards washed with cement grout. The writer was informed that extreme haste was used in laying the pipe and that the joints are very rough internally. This would account for the heavy friction loss in this pipe. Examination of both inlet and outlet indicated that this pipe was free from gravel or other débris. At the time of these tests the water was murky from recent rainstorms and the spring thaw, but during most of the irrigation season the water is clear. The value of C_s is about 0.255.

No. 17, Experiment S-46.—20-inch jointed concrete pipe, Bishop lateral, British Columbia Fruitlands Co., Kamloops, British Columbia, Canada.—Irrigation water is diverted from the main concrete-lined canal by a tube through the canal bank, discharging into the sand and weir box shown on Plate II, figure 1. From this weir the Bishop lateral, an underground pipe line, leads down a steep hillside to a point near the bank of Thompson River, thence paralleling the river. A reach of pipe between a vertical air pipe and a turnout box was chosen for experiment. The airpipe was located about three-eighths mile from the weir shown in the plate, while the turnout was 777.1 feet beyond the air pipe. Water columns attached to piezometers of type A were used at both ends of the reach, No. 1 being set 4 feet down the pipe line from the air pipe and No. 2 being thrust 1.3 feet upstream into the pipe line from the turnout. The pipe is gently curving in horizontal alignment and appears to follow the river grade in profile. If there is a sag in the profile it is too small to be noticeable to the eye. Examination of the line when no water was flowing showed the interior to be clean of deposit, but typically rough to the touch in the usual washed-cement pipe way. The discharge was measured over the rectangular, contracted weir shown in the plate (fig. 1). The crest is 3.02 feet long, the end contractions each 1 foot, and the bottom contraction more than 3 feet. For two observations the end contractions were very slightly less than twice the depth on the weir crest; for the others the end contractions exceeded twice the depth. Elevations of the water surface above the weir crest were determined with the hook gauge shown in the stilling box. The mean inside area of the pipe line was taken as the average area of 10 samples of the pipe remaining from construction. This area was 2.106 square feet, while the area of a true 20-inch pipe is 2.182 square feet. Thus this pipe is 3.5 per cent under size. The velocity of the water in the pipe was found by dividing the discharge in second-feet by the area of the pipe in square feet. This line was laid in 1911, thus being in its fifth year of operation at time of test. The coefficient $C_s=0.313$.

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[1] Described in "Irrigation Practice and Engineering," Vol. III, p. 368, B. A. Etcheverry, New York, 1916.

No. 18, Experiment S-60.—20-inch jointed reinforced concrete pipe, Temescal Water Co., Corona, Calif.—Water for irrigation and domestic use is conveyed through 27,000 feet of plain concrete pipe flow line 22 inches in diameter and over 15 depressions which require inverted siphons.[1] Of these, 13 are of reinforced concrete, 20 inches in diameter. One of the longest of the siphons, about 4 miles from Corona, was chosen for experiment. Open standpipes occurred near both inlet and outlet of the siphon. Water columns attached to piezometer tubes of type A were used at both ends of the reaches chosen. For observations 1, 2, and 3, piezometer tube No. 1 was thrust 9.8 feet down the line from the standpipe at the inlet. For observations 4 and 5 a hole was tapped in the top of the concrete pipe 101.7 feet from the intake manhole and the piezometer tube thrust 6 feet farther down the pipe. For all observations piezometer No. 2 was set in the pipe 0.8 foot upstream from a hole tapped in the concrete pipe 98.7 feet upstream from the manhole at the outlet. Velocities were ascertained by the use of solutions of potassium permanganate and of fluorescein. For observations 1, 2, and 3 the color was injected in the manhole at the inlet, while for observations 4 and 5 it was injected at the tap in the concrete pipe 101.7 feet from the manhole. For all observations the color was observed at the manhole near the outlet. This pipe was cast on the ground in units 3 feet long. For moderate heads a 1 : 2 : 4 mixture of cement, sand, and gravel was used and for higher heads the mixture was changed to a ration of 1 : 2 : 3. All units were "washed" with cement grout before being laid. This siphon is straight in horizontal alignment but is on practically one long double-reverse vertical curve, the maximum head being about 80 feet, while the total length from manhole to manhole is 2,273 feet. The water is very clear and this siphon is undoubtedly absolutely free from débris, as all sand must pass more than 15 other siphons before reaching this one, the flow line between this siphon and the next one upstream being but 300 or 400 feet long. One blow-off valve is located at the low point of the siphon. The pipe was laid early in the year 1911 and was thus about 5½ years old at the time of the experiment. So far as could be determined from the low standpipes, the joints were well smoothed and this pipe is probably typical of present-day careful practice in the citrus groves of southern California where long, well-made units are being used. The value of C_s is 0.345.

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[1] Engin. Rec., vol. 64, Nov. 4, 1911, p. 526.

No. 20, Experiment S-57.—24-inch monolithic pipe, Clavey siphon, Oakdale irrigation district, California.—As described under No. 28, the first portion of the Clavey siphon is 36 inches in diameter from the inlet to a "booster pump" which is used to force part of the water through 3,000 feet of 24-inch pipe to a point 8 feet higher than the inlet to the siphon. Tests were conducted on a reach 1,046.2 feet long between an air valve and a point 6.6 feet upstream from the outlet of the 24-inch pipe. Water columns attached to piezometer tubes of type A were used for both gauges. Velocities were determined with potassium permanganate solutions injected at gauge 1 and observed at the outlet. The color dragged out from 6.2 to 9.6 per cent of the time from moment of injection to the mean time of first and last appearance at the outlet, depending on the intensity of the injection. Although nominally a 24-inch pipe, the mean of 6 measurements made in the last 30 feet of the pipe indicates the actual diameter was 23.7 inches, which was used in computations on this pipe. This pipe was constructed with wood forms, both inside and out. The longitudinal ridges made by the cracks in the boards, the circular ridge at the end of one setting of the forms and cemented concrete scraps are shown clearly in Plate II, figure 2, which was photographed from a point about 20 feet in the pipe at the outlet end. The values of C_s are erratic, varying from 0.328 to 0.373.

No. 21, Experiment S-47.—24-inch jointed concrete pipe, lateral A, British Columbia Fruitlands Co., Kamloops, British Columbia, Canada.—A reach of lateral A pipe line 2,306.5 feet long was chosen for test. Water entered the pipe from a box similar to the one described under No. 16. Likewise the installation of the piezometers was identical with Nos. 16 and 17. The first standpipe was located 64.7 feet from the inlet of the pipe. Between the two gauges were located two turnout boxes (not in use at time of tests), one 12-inch standpipe, and one waste valve. Water in the standpipe rose to the hydraulic gradient. The writer had been notified that the pipe was partially filled with gravel from the hillside above the main canal. A heavy rainstorm had eroded the hill, filled the main canal near the inlet and regulating box with gravel, and washed some of the latter into the pipe line. The ratio between the measured size of several samples of this pipe and the true area of the water section was found in the following way: For one particular observation the velocity of the water through the reach was determined with two injections of color—one permanganate and one fluorescein. These agreed within 2 seconds in an elapsed time of 1,800 seconds. The quantity of water was determined by hook-gauge readings on the 3-foot contracted weir at the pipe inlet. The area was then found by solving the equation $A=\frac{Q}{V}$. The area thus found was 2.754 square feet or 87.5 per cent of the nominal area of a 24-inch pipe and about the equivalent of the area of a pipe 23.5 inches in diameter. The mean value of C_s is 0.252.

No. 22, Experiment S-61.—30-inch jointed concrete pipe, Temescal Water Co., Corona, Calif.—Water for irrigation and domestic use is conveyed over a shallow depression by a siphon pipe 1,028 feet long, inserted between an open concrete-lined channel in sandy soil and a round pipe flow line. The siphon pipe is made of precast units, each 3 feet in length. The pipe is practically straight in both horizontal and vertical planes, the maximum head being about 8 feet. This line was constructed in 1911, being thus in its fifth year of operation at the time of the experiment. It carries water throughout the year. A reach was chosen for test between a point 79 feet from the intake chamber and a point 69 feet upstream from the outlet manhole. Water columns attached to piezometers of type A were used at both ends of the reach. Velocities were determined by the use of solutions of potassium permanganate, injected at gauge No. 1 and observed at the manhole beyond gauge No. 2. Measurements of the initial pipe section at the intake chamber showed the diameter to be 2.49 feet, or 0.01 foot less than the nominal diameter.

No. 23, Experiment S-34.—30-inch Jointed Reinforced Concrete Pipe, "D_1 Line", Umatilla project, United States Reclamation Service, Oregon.—Water for irrigation is conveyed across the depression near Hermiston, Oregon, in a 30-inch pipe laid in sections, each 4 feet long.[1] This line was constructed in the winter of 1909–10 and tested for friction losses in 1911 by Mr. H. D. Newell.[2] The writer conducted tests on the same reach of pipe during the season of 1915. See No. 12a (page 77) and No. 24a, for pipe description. Thus while comparatively new when first tested it had been in use nearly six seasons when these experiments were made. The pipe units were each 4 feet in length, cast in wood forms that had been coated with No. 26 sheet steel. The mixture and wash coating is described under pipe No. 32, page 41. Gage No. 1 was a piezometer tube of type B inserted into the water section of the pipe through a stuffing box, located above the valve A in figure 3, Plate 2. Newell appears to have used the pressure directly from a $\frac{3}{4}$-inch valve set in the pipe line. The writer found so much air in the water column taken directly from this tap that he judged a piezometer tube, even of type B, would give more nearly the true pressure head (see discussion on page 15). This tube was connected with a mercury manometer. At the outlet a piezometer tube of type A was thrust 3.6 feet into the pipe, against the current. This tube was connected with a stilling box and the water surface in the latter was read by a hook gauge. The relation of the points plotted on Plate VI for the Newell tests (No. 24) and for those made by the writer (No. 23) indicate that the capacity of this pipe has diminished slightly. This was to be expected, for Newell speaks of deposits in the pipe after but one season while the writer's tests were made in the sixth season. Until July of each year the water for this pipe comes directly from the Umatilla River, through about 4 miles of open canal, the last reach of which discharges in a direct line into the pipe intake. This river water, taken during the high-water period, would undoubtedly contribute greatly toward silt and débris within the pipe as the maximum demand so far has not required a velocity in the siphon in excess of about 2.5 feet per second. Velocities were determined directly with solutions of fluorescein, the mean time of several batches being accepted in computing the mean velocity. This pipe is quite straight in horizontal alignment, while the vertical curves are long and gentle. There are five 6-inch valves and three manholes on the reach tested. Examination of the outlet indicates that the pipe is smooth and slimy inside. The color was injected into the pipe line between the piezometer tube and the larger iron pipe (see Pl. II, fig. 3).

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[1] Eng. News, Feb. 16, 1911, Vol. 65, p. 208.

[2] Id. May 1, 1913, Vol. 59, p. 904.

While but three observations were made on this pipe the conditions were very favorable for test. The values of C_s, about 0.330, are probably indicative of the silted condition within a smooth pipe rather than of the interior surface itself. (See values for Newell's tests, No. 24*a*.)

No. 26, Experiment S-51.—30½-inch jointed reinforced concrete pipe, Prosser pressure pipe, Sunnyside project, United States Reclamation Service, Washington.—Irrigation water is conveyed across the valley of the Yakima River by a siphon pipe of combination type. From the intake to a point 2,825 feet distant concrete pipe is used. The construction then changes to a wood-stave pipe[1] 31 inches in diameter, which extends for an additional distance of 7,500 feet. A straight reach of the concrete pipe 2,276.3 feet long was chosen for test. Gauge No. 1, a water column attached to a piezometer tube of type A, was located about 500 feet from the intake, while gauge No. 2, a mercury column attached to a piezometer of type C, had been installed at the time of construction. A similar connection of type C had been placed near the upper end of the line, but was destroyed by the construction of a railroad crossing. It was necessary, therefore, to drill a new hole through the pipe and use a connection of type A. The velocities were determined by timing solutions of fluorescein, injected at gauge No. 1 and withdrawn into a white-lined pan from a secondary connection to the pipe line located near the head of the wood-stave portion of the line, beyond the end of the concrete pipe. The slight difference in the area of the wood and the concrete line was considered in computing the velocity in the concrete line alone. The mean area of the pipe was taken as the mean of the areas of six units of the pipe remaining from construction. It is quite certain that there was no sediment in the pipe, as the reach tested is on a continuous down grade and the operating velocities high enough to carry any débris on down to the sag of the siphon, which occurs in the wood-stave portion of the line several thousand feet beyond the concrete pipe. Examination of the line during the series of tests disclosed the scour at the upper end of the line (mentioned on p. 51), but showed a pipe free from slime or deposit at the upper end of the reach tested, and this condition undoubtedly held throughout the reach. The joints were not quite as nearly perfect as on the Victoria line (Nos. 30 and 31), and this probably accounts for some of the discrepancy in relative carrying capacities. As gauge No. 1 is located at an elevation approximately 20 feet higher than that of the outlet, it is readily understood that only velocities higher than that which necessitates a loss of head of 20 feet for the 2 miles of line beyond gauge No. 1 may be tested, because for all lesser velocities the pipe is not full at gauge No. 1. All concrete lines that serve as the intake ends of combination pipes are thus not subject to a complete test throughout a range of velocities, say, from 1 foot per second up, but can only be tested for the higher velocities.

The concrete pipe was made in 4-foot lengths with an exceedingly wet mixture of 1 part cement to 6 of gravel. All gravel passed a quarter-inch mesh screen. Steel forms coated with a particularly heavy brick oil gave a smooth surface to the interior of the pipe. The concrete was tamped into the molds, and just before the initial set was retamped by a finisher, settling in the forms about 3 inches.

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[1] Discussed as No. 35 in Bul. 376, U. S. Dept. Agr., p. 78.

This line was designed by use of the Kutter formula, with a value of n taken as 0.013. The maximum capacity of the whole siphon was computed as 26.6 second-feet; but, whereas 29.39 second-feet was sent through the pipe during observation No. 2, this discharge practically filled the intake chamber and may be taken as the maximum capacity of the pipeline at the present time.

Since there is a long stretch of wood-stave pipe beyond the concrete pipe, each slight change in the velocity through the latter made a great change in the heights of the piezometer columns. A velocity of 4.88 feet per second was the lowest that could be tested, as it developed but a slight pressure head at the upper end of the reach. The velocity for maximum capacity was 5.78 feet per second. Thus the range of velocities feasible was rather short. Reference to Plate VI shows that, while consistent, the arrangement of the observation points indicates a very high exponent of V, namely, 3.18. Had the full range of velocities—between, say 1 foot per second and 5.78 feet per second—been possible of test, the points of observation might have more nearly compared to a slope of 2. Otherwise the conditions of experimentation appeared very good, and the writer is at a loss to explain the behavior of the points.

The friction factors, by the same token, vary, the coefficient C_s changing between 3.86 and 3.43, while from the appearance of the pipe interior it should have been about 0.360 for all observations.

Nos. 28 and 28b, Experiment S-54-55.—36-inch monolithic pipe, Clavey siphon, Oakdale irrigation district, California.—The Clavey siphon of this district, built in 1912, is of unusual construction. Water from an open channel lateral is conveyed 3,000 feet across a swale in a 36-inch reinforced concrete pipe, built in place. The outlet end is a few feet lower than the inlet. Thus far the construction is as usual.

Near the bottom of the swale, a few hundred feet before reaching the outlet, water from this pipe may be diverted through a booster pump. The energy of the total flow in the 36-inch line is used to pump a part of this same water through an additional 3,000 feet of 24-inch pipe to a point about 8 feet higher than the inlet of the siphon..

Both parts of this siphon were constructed in the same manner. Wood forms were used for inside and outside surfaces. This construction usually gives longitudinal ridges in the concrete where each crack between boards occurs and more or less irregularity in the surface at the abutting sections. This is brought out in Plate I, figure 1, and Plate II, figure 2. The mixture used was 1 part of cement to 6 parts of gravel. The resulting concrete is not first class, there being several leaks in the siphon. Most of these leaks occur at the joints between abutting ends of pipe sections. Two reaches of the 36-inch pipe were chosen for tests, No. 28*b* being 1,266 feet long and No. 28 being 1,933.6 feet long, but including all of reach No. 28*b*. Two lengths were tested, as the inlet end of the pipe was not completely filled during observations 1 to 3, inclusive, because of the small quantity of water, and the fact that the booster pump was not in operation, the longer reach of pipe being tested when the booster pump was lifting water to the outlet of the 24-inch pipe. On reach No. 28*b* a water column was used as gauge 1 and a mercury manometer as gauge 2. On reach No. 28 the water column was moved 667.6 feet nearer the inlet, but the mercury manometer remained the same as before. Air troubles at gauge 1 were minimized by the device shown in Plate V, figure 1. Piezometer tubes of type A were used for both gauges, No. 1 being slipped into the pipe through a one-eighth-inch wrought-iron nipple, while No. 2 was thrust down an air valve. Velocities were determined by timing fluorescein from its injection at gauge 1 to its appearance at gauge 2. For drawing off the colored water a "gooseneck" of one-fourth-inch brass pipe was inserted down the air valve and the color detected in a white-lined pan.

For some reason that the writer is not able to explain the friction factors are erratic and inconsistent.

No. 29, Experiment S-36.—36-inch jointed reinforced concrete pipe, Deer Flat Forest pipe line, Boise project, United States Reclamation Service, Idaho.—Water for irrigation is conveyed across a wide depression, just below the dam of Deer Flat Reservoir, in a concrete siphon pipe 8,575 feet long and 36 inches inside diameter.[1] As shown in Plate IV, figure 1, this line is straight in horizontal alignment and without vertical curves other than two gentle bends at the bottom of the slopes near the inlet and outlet. The maximum head is about 70 feet.

The pipe units, 6 feet in length, were cast on the ground in steel forms. A very wet mixture of 1 part cement to 2¼ parts sand and 3 parts well-graded gravel resulted in a dense concrete. The shell is 3 inches thick, reinforced with five-sixteenth-inch wire. As shown in the plate, the joints were made with reinforced collars, each 3 inches thick and 8 inches wide in addition to the usual bevel and taper. The joints were calked on the inside with great care, the mixture used being 1 part cement to 2 of sand and tempered with hydrated lime in the proportion of 10 per cent of the cement by volume.

A reach of this pipe 7,282 feet long from the foot of the first slope to the outlet was chosen for test. Gauge No. 1 was a mercury manometer attached to the pipe at a small hole through a cast-iron manhole cover. At the outlet a piezometer tube of type A was thrust 2 feet into the pipe, and the pressure head conveyed by tubing to a stilling box within the outlet chamber, the elevation of the water in the box being determined by a hook gauge.

Unfortunately, in 1915 it was not feasible to vary the discharge through the pipe, so only one observation could be made. The velocity of the water was determined by accepting the mean of five batches of fluorescein, injected at gauge No. 1 and observed at the outlet. Accepting the mean diameters of 6 units of pipe remaining from construction, 2.999 feet, as the mean diameter of the pipe line, then the discharge was 24.6 second-feet, while the mean discharge over a weir below the outlet was 24.3 second-feet and the discharge as measured by a Price current meter, using the integration method, was 24.57 second-feet, and using the 0.2 and 0.8 depth method was 25.15 second-feet (see Table 2, p. 18).

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[1] Engin. News, Aug. 8, 1912, vol. 68, p. 248.

The large pool just above the 10-foot Cipolletti weir was cleared of sand just prior to the experiments. The end and bottom contractions were equal to about twice the depth of water over the crest. The velocity of approach was about 1 foot per second. More exact figures were not feasible, due to shape of pool. The current-meter measurements were made by the hydrographer of the project simultaneously with the reading of gauges and the passage of the color in the pipe.

During the summer of 1917 the writer again visited this pipe and made additional tests on approximately the same reach of pipe. There being an abundance of water it was feasible to vary the discharge through the pipe and thus secure tests at various velocities. Gauge No. 1 was attached to a piezometer of type A while the piezometer at gauge No. 2 was very similar to that in the 1915 tests, but a glass gauge tube was used instead of a stilling box.

The velocities were determined by the travel of fluorescein solutions, which were timed from the intake of the siphon to the outlet. The average value of C_s for all the tests was 0.395. The high velocities probably assure a pipe free from sediment and the conditions at the intake are such that no gravel or heavy detritus would enter the siphon. As may be seen from the photograph, this pipe is exceptionally straight and under most favorable flow conditions. The writer would consider the coefficients found as verifying a working coefficient of 0.370 for glazed pipe under average conditions of curvature.

No. 30, Experiment S-49.—42-inch jointed reinforced concrete pipe, Victoria Aqueduct, siphon No. 1, Vancouver Island, British Columbia, Canada.—A pipe line quite typical of present-day high-grade construction has recently been built to convey water for municipal purposes from Sooke Lake, a mountain reservoir, to Humpback Reservoir, about 10 miles from Victoria, British Columbia.[1]

This line, 27.3 miles in length, follows the hydraulic gradient as a flow line with the exception of 6 inverted siphons. The slope throughout is 0.001 foot per foot or 1 foot fall per thousand feet of length. Slightly more than one-half the line is curved. The pipe units are of the lock-joint type, each 4 feet long and exactly 42 inches in diameter. The shell of the flow line is 3 inches thick and that of the siphons 4½ inches thick. The maximum head on any of the siphons is 90 feet. The flow-line pipe is reinforced with triangular mesh wire and square steel bars are used in the pressure units. The concrete, a "wet mix," composed of 1 part cement, 2 parts clean sharp sand, and 4 parts of beach gravel (maximum diameter three-fourths inch), was tamped into steel forms with a suitable slice bar, care being taken to force the coarse aggregate away from the forms, thus making a smoother surface.

The forms were well wiped with oil just prior to each pouring. After being filled they were placed in a steam room for three hours; the forms were then stripped and the pipe, resting on a base ring, again subjected to steam for an additional four hours. The interior surface of pipes made in this way is unusually smooth. At times there was small pitting, the cause of which remained obscure. If too noticeable such a surface was wiped with thin mortar. Unless pitted no wash coat was applied.

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[1] Engin. Rec., vol. 69, Feb. 21, 1914, p. 225; *id.*, vol. 72, Oct. 2, 1915, p. 406; *id.*, vol. 72, Oct. 23, 1915, p. 507. Can. Engr., July 23, 1914, vol. 27, p. 197; *id.*, June 10, 1915, vol. 28, p. 652. Western Engineering, vol. 5, Sept. 1914, p. 105.

The great amount of curvature was cared for by making three types of bevel-end sections and making the degree of curvature for each curve on the line such that one of the three types could be used with a maximum bend of 3° at each joint. This limit could be attained without inducing appreciable roughness within the pipe.

As shown in Plate VIII, figure 1, the various sections of pipe were fitted together on the ground without the use of cement, a "soup hole" being broken out of the top of each section just before fitting the following section. The outside of each joint was banked up with clay or stiff gravel to a point well above the spring line, in order to prevent loss of grout. In cementing the joints the invert was troweled from the inside. Then a band of spring steel 6 inches wide was tightened against the interior of the joint and grout poured into the soup hole from a coal bucket. When the grout had set the band was removed and any necessary smoothing done with a trowel. The writer examined the interior of this line and found the joints as smooth to the touch as the rest of the pipe. Although this line is not covered with earth, no expansion joints were inserted at the time of construction, it being thought that the great amount of curvature would automatically care for temperature changes. Experience with this line during the winter of 1916–17 shows that it would have been better to insert an expansion joint every few lengths of pipe.

Siphon No. 1, between manholes 4 and 5, was chosen for the major tests upon this line. It is longer than any of the other siphons and near enough to the gatehouse so that alterations in discharge may comparatively quickly reach the section under test.

Holes were drilled through the top of the pipe near each end of the siphon at such elevations that the pipe was completely filled at these holes regardless of discharge. Piezometer tubes of type A were thrust into the siphon through these holes. Water was turned out of the pipe while the piezometers were carefully placed on the inside. These tubes were connected with gauge glasses by means of which the pressure head was read directly in water columns.

Velocities were determined by fluorescein injected into the hole in the concrete pipe made for piezometer No. 1 and observed through the manhole in the flow line at the end of the siphon, beyond the hole for piezometer No. 2. A correction was necessary for the reason that the water did not fill the pipe for a few feet between gauge No. 2 and the manhole. Data for this correction were carefully taken with the level at the time the levels were run for the determination of the loss of head.

As stated in the discussion under No. 54*a*, page 86, there is a weir just above the intake to the pipe line. The crest is about one-fourth inch thick, rounded over on a radius equal to about one-half the thickness of the plate. Water entered the chamber above the weir from one side, making an indeterminate condition of approach velocity. However, a float gauge was read for each run of water and the discharge computed by the Francis formula. The comparison of the velocity in the siphon pipe as computed from this weir discharge and as determined by the direct timing of color is given in Table 2, page 18. It is to be noted that, for all runs, the velocity by direct measurement is greater than as computed by weir discharge.

In other words, as is to be expected from the conditions, the weir discharges more water for a given head than a standard Francis weir would have done. As mentioned above, the velocities used in the computations in tables 3 and 4 were all based on the color method.

The mean diameter of this pipe, found by measuring two diameters on each of nine pieces remaining from construction is 3.50 feet, agreeing with the nominal diameter. All things considered, the writer regards this series as among the best, due to the very favorable conditions for experimentation. The value of C_s is about 0.375 and may be taken as for a very smooth pipe with an excess of curvature.

No. 31, Experiment S-48.—42-inch jointed reinforced concrete pipe, siphon No. 5, Victoria Aqueduct, Vancouver Island, British Columbia, Canada.—This series of tests was conducted on the last of the siphons on this pipe line, siphon No. 5, located between manholes 67 and 68. The same general discussion applies to this series as to No. 30. This siphon is much shorter, and in consequence the series should not be as reliable as No. 30. As shown in Plate V, figure 3, the siphon pitches down a steep grade, thence extends horizontally over a concrete trestle, thence climbs a steep hillside to the outlet manhole (in the foreground).

The values of C_s being around 0.390 would indicate a higher capacity than the observations on pipe No. 30, but those should be accepted over the tests on siphon No. 5.

No. 32, Experiment S-35.—46-inch jointed reinforced concrete pipe, R_1 siphon, Umatilla project, United States Reclamation Service, Oregon.—Water for irrigation is conveyed across a wide valley, from one open channel to another, by means of an inverted siphon pipe 9,830.8 feet long and subject to a maximum pressure head of 110 feet. It was constructed in the winter of 1909–10. The joint units are 8 feet in length, reinforced with a spiral of five-sixteenth-inch steel wire wound round longitudinal rods (Pl. VIII, fig. 2). The concrete was mixed in a ratio of 1 part cement to 1.44 parts sand and 2 parts gravel. The units were cast in steel forms and afterward painted on the inside with a grout made by pouring one-half bucket of cement into two-thirds of a bucket of water. This made a full bucket of a mixture of about the consistency of cream.[1]

As shown in Plate VIII, fig. 2, the pipe is straight in alignment, while the profile is evident, with the exception of the deep sag across the lower lands, where the pressure head reaches 110 feet. Within the reach of pipe tested there are eight 6-inch valves, two 6-inch blow-offs, and four manholes, each 12 by 14 inches.

This line was tested by Mr. Newell in 1911 and again in 1912 (see No. 33*a*, p. 79). During the season of 1915 the writer conducted experiments on the same reach of pipe, between a valve 72 feet from the inlet and the outlet chamber, a distance of 9,774 feet.

For gauge No. 1 a mercury column was used, while for gauge No. 2 a piezometer tube of type B, extending into the jet at the outlet, led to a stilling box wherein the water-surface elevation was determined with a hook gauge. Two diameters were measured on each of 8 pipe units remaining from the original construction. The mean area of these units was 11.52 square feet, while the exact area of a 46-inch pipe is 11.54 square feet. The diameter of the pipe line was accepted as 46 inches.

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[1] The details of manufacturing experience and costs covering a period of about three years are given in "Reinforced-Concrete Pressure Pipe on the Umatilla Project, Oreg., U. S. Reclamation Service," by H. D. Newell, Eng. News, vol. 65, Feb. 16, 1911, p. 208.

The velocities for the various runs were determined by timing the passage of fluorescein from the point of injection, at gauge 1, to its appearance at the outlet. From 2 to 5 batches of color were used for each run. For any one run the elapsed time of the colors agreed within one-third of 1 per cent.

Reference to Plate VI shows that the curve for the 1915 experiments, when produced, passes through the midst of the points for Newell's tests made in 1911 and 1912. If this fact is truly indicative, then the capacity has not changed during the interim. This is reasonable, because the pipe is operated at a rather high velocity throughout the season and the feeder canal leading from the main canal to the intake is in a hard cemented gravel that would not contribute ravelings or silt, while the water itself comes from Cold Springs reservoir, and so is free from silt, except such as it may pick up in transit from the reservoir to the pipe. The clean condition of the pipe interior, where examined at the outlet, would indicate a pipe comparatively free from slime.

The mean value of the coefficient in the new formula, as determined from the Newell tests and our own, is 0.395, but it must be borne in mind that this pipe is practically straight, a rather unusual condition for a pipe nearly 2 miles in length.

No. 33b, Experiment S-60.—48-inch monolithic reinforced concrete pipe, Churn Creek siphon, Anderson-Cottonwood irrigation district, California.—In order to serve a certain acreage of land lying east of the Sacramento River, the flow of Churn Creek lateral is conveyed for about 4,300 feet in a round, monolithic siphon pipe extending from an open earth canal to a steel flume across the main channel of the river. (Pl. IV, fig. 2.) It was the original intention to extend the siphon under the bed of the river as a cast-iron pipe, but this plan was changed, so that at the edge of the deep-water channel the concrete pipe is turned vertically upward and placed in the first pier of a bridge across the river. This bridge supports a steel flume and the pipe discharges directly into the flume.

As shown in the plate, the siphon was constructed continuously, wood being used for both inside and outside forms. Though originally designed 4½ feet in diameter, the pipe was constructed with a diameter of but 4 feet, owing to a reduction in the acreage to be served.

A single experiment for loss of head was made on this pipe in the summer of 1919. Piezometer tubes of type A were thrust into the siphon at inlet and outlet and the pressure head was carried over the banks to glass-gauge water columns. Piezometer No. 1 was thrust about 6 diameters (24.7 feet) down the inlet end and No. 2 was held 10 feet down the vertical outlet pipe located in the bridge pier. Between these piezometers there is a developed distance of 4,242.3 feet, so that a very material loss of head occurs for the commercial velocities—between 3 and 4 feet per second. The reach tested is practically straight in both vertical and horizontal planes, with the exception of the 90° bend as the pipe is turned upward in the bridge pier. The loss due to this bend is immaterial compared to the friction loss, and was ignored in the computations. The velocity of the water in the pipe was determined by injecting a solution of "Congo red" dye directly into the mouth of the siphon pipe, not into the pool in the intake chamber. It was 1,091.3 seconds before the color appeared at the outlet and 84 seconds more before the last trace disappeared.

In computing the velocity the elapsed time is taken as from the moment of injection of the solution to the mean between first and last appearances of the color at the outlet. Being injected under a pneumatic pressure of 60 or 70 pounds against a static head of but 2 or 3 pounds, the "shot" of color leaves the color gun in but a fraction of a second, the lever handle of the valve being turned across the orifice slowly but continuously. When tried out in an open channel, where the injection can be watched, a great cloud of dense color, 3 or 4 feet in diameter, can be shot into the water, with an instantaneous opening and closing of the valve, merely by turning the handle through 180°.

As could be seen in the photographs from which the plate was made, each crack between the boards of the forms was clearly defined in the concrete structure. No opportunity has offered for examination of the interior of the siphon, but the velocities are such that little or no silt should collect in the pipe. The retardation factor of 0.313 confirms the recommendation of 0.310 for pipes of similar construction.

No. 35, Experiment S-40.—63½-inch monolithic reinforced concrete pipe, Simms Creek siphon, Sun River project, United States Reclamation Service, Montana.[1] Water for irrigation is carried over Simms Creek in a siphon pipe inserted between open channels.

This siphon was constructed in the winter of 1907–8. It was built in place, steel in 6-foot sections being used for the interior and wood for the outside forms. The inside was then washed with cement grout. From intake to outlet the pipe is 1,556 feet long and 5.3 feet in diameter.

Preparatory to the experiments conducted by the writer, piezometer connections of type C (fig. 4) were set in the zenith of the line at distances of 40 and 400 feet from the intake and 20 feet above the outlet. These will be referred to as taps 1, 2, and 3, respectively. At the time these taps were set the project manager inspected the pipe and found the interior in a very smooth, clean condition. So far as he is aware, the pipe had never been cleaned, although in its eighth year of operation.

The loss of head for the various velocities was measured between taps 2 and 3, a mercury column being used at tap 2 and a stilling-box type of water column at tap 3. In order to fill the pipe completely at tap 3 it was necessary to insert flashboards in the slots of the outlet structure. The velocity was determined by accepting the mean time of two or three batches of fluorescein. For observation 1 the color was injected at tap 2, but for the other runs the color was injected at tap 1. The first and last appearance was noted at the outlet of the siphon. Thus for all observations other than No. 1 the color traversed a reach of 379.5 feet longer than the reach between manometers, while for run 1 the color reach was but 20 feet longer. The line was designed for a capacity of 175 second-feet of water, based on a value of *n* in the Kutter formula of 0.012. Mr. C. P. Williams, project manager, stated to the writer that "The maximum amount that had been carried by the siphon was about 225 second-feet. At this time there was 255 second-feet diverted into the head of the canal and the loss between the head and the siphon was about 30 second-feet." Our experiments also bear out the fact that the capacity is greater than a value of n of 0.012 would indicate. Air troubles in gauge No. 1 caused inconsistent results for this series, and for this reason full weight should not be assigned to the tests. The values of C_s range from 0.380 to 0.434, and the corresponding values of n from 0.0116 down to 0.0101. During the season of 1917 the writer again visited this pipe while it was carrying far more water than for any of the observations made in 1915, but the air troubles were even greater than before, so experimentation was considered out of the question. This pipe shows the need of "air chimneys" near the intake if maximum efficiency is to be reached.

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[1] Eng. Rec., vol. 59, 1909, p. 716.

No. 37, Experiment S-59.—120-inch monolithic concrete pipe, Whitney siphon, Los Angeles Aqueduct, California.[1]—The Whitney siphon, on the Saugus division, is 955 feet long, 10 feet inside diameter, with a shell 9 inches thick, and is subject to a head of about 75 feet. It conveys water for municipal and irrigation purposes across a narrow canyon, between two flow-line tunnels. The sides of the canyon have a slope of about 3 feet horizontal to 1 foot vertical.

The pipe was constructed in place, smooth wood forms being used for both inside and outside surfaces.

The interior of the pipe was treated with a finish coat of rich cement mortar. The grit of the sand in this coat was very noticeable to the touch at the time of experiments in 1916.

A reach of the siphon pipe below the level of the floor of the flow-line at both inlet and outlet was chosen for test. Holes were drilled through the top of the shell and one-eighth-inch iron-pipe nipples, each 18 inches long, were cemented into the shell. (The hole for a one-eighth-inch pipe is nearly one-fourth inch in diameter.) Two piezometer tubes of type A, exactly alike in construction, were thrust into the pipe through the nipples. The tapering end of these tubes was made flexible by a small rubber-hose joint with the main tube, so that the current might hold each piezometer in the direction of flow by means of the flattened taper, like a vane.

The flow-line channel is tapered into the round section of the siphon pipe by means of transition sections at each end of the siphon. Between the transition sections and the flow-line sections manholes are placed at both inlet and outlet.

The tap for gauge No. 1 piezometer was 59.2 feet downstream from the manhole at the inlet, while the tap at gauge No. 2 was 61.1 feet upstream from the manhole at the outlet. The holes in each piezometer tube were 3.6 feet downstream from the taps in the pipe. This made the reach between piezometers 857.4 feet long. Water columns were used at both gauges.

The nominal size of the pipe was accepted in the computations. The velocities were determined by timing the passage of solutions of fluorescein, injected at gauge No. 1 and observed at the manhole beyond gauge No. 2, correction being made for the fact that the channel is not full for part of the distance between gauge No. 2 and the manhole.

Experience with this pipe showed that satisfactory tests can not be made on a reach of pipe that is relatively short, compared with the size of the pipe, especially for low velocities. The loss of head for

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[1] Construction of the Los Angeles Aqueduct, Final Report, Los Angeles, Calif., 1916, pp. 210, 214. Engin. Con. July 3, 1912, p. 20.

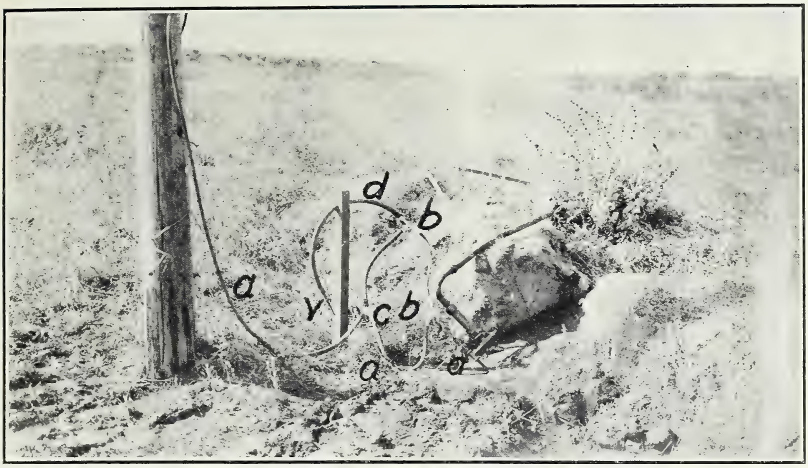


FIG. 1.—CONNECTION AT GAUGE 1, CLAVEY SIPHON, OAKDALE IRRIGATION DISTRICT, CALIFORNIA. (PIPE NO. 28.)

Note air trap and color reservoir. Gauge glass is on pole.

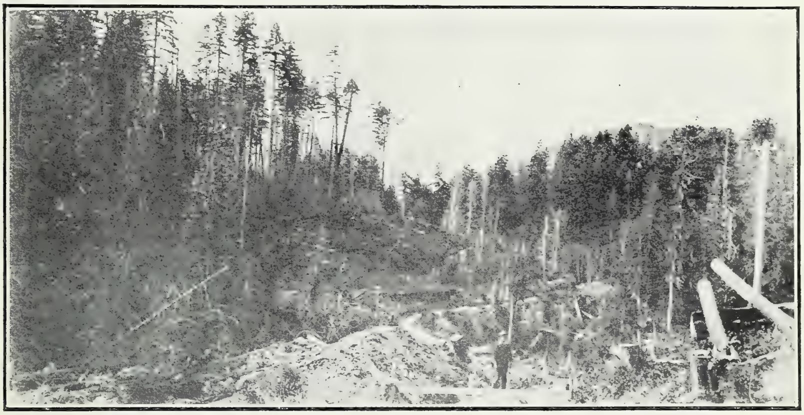


FIG. 2.—SIPHON NO. 1, VICTORIA AQUEDUCT. (PIPE NO. 30.)

Mr. Ewing is standing at tap for gauge No. 1.

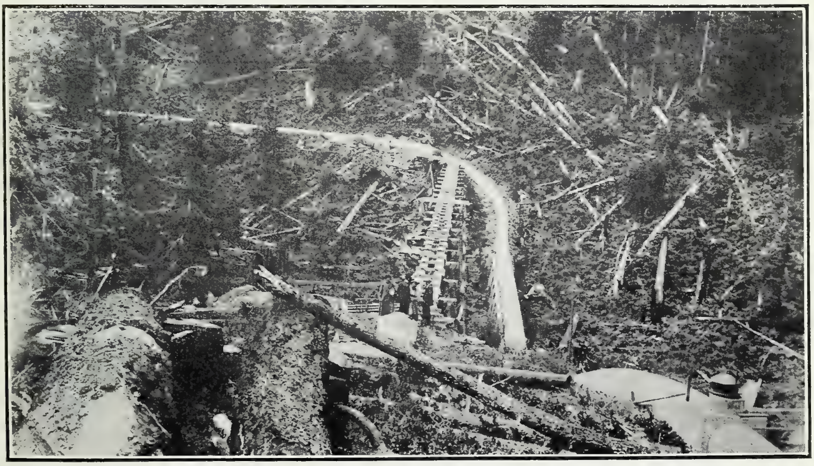
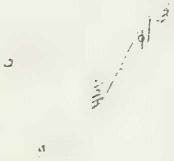
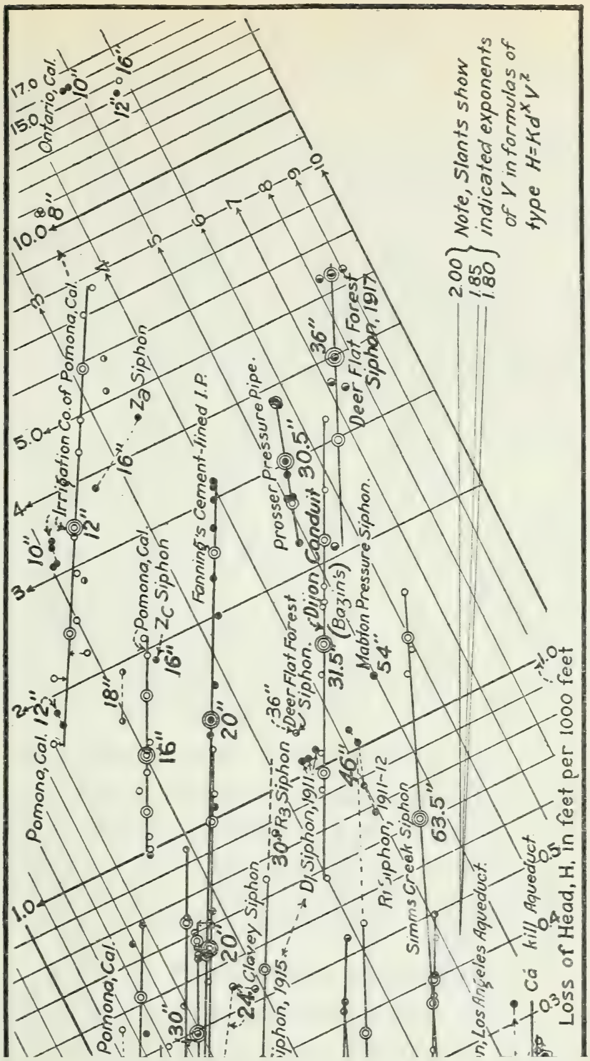


FIG. 3.—SIPHON NO. 5, VICTORIA AQUEDUCT. (PIPE NO. 31.)

Note manholes and concrete trestle over creek. Wood trestle used during construction for conveyance of pipe units.

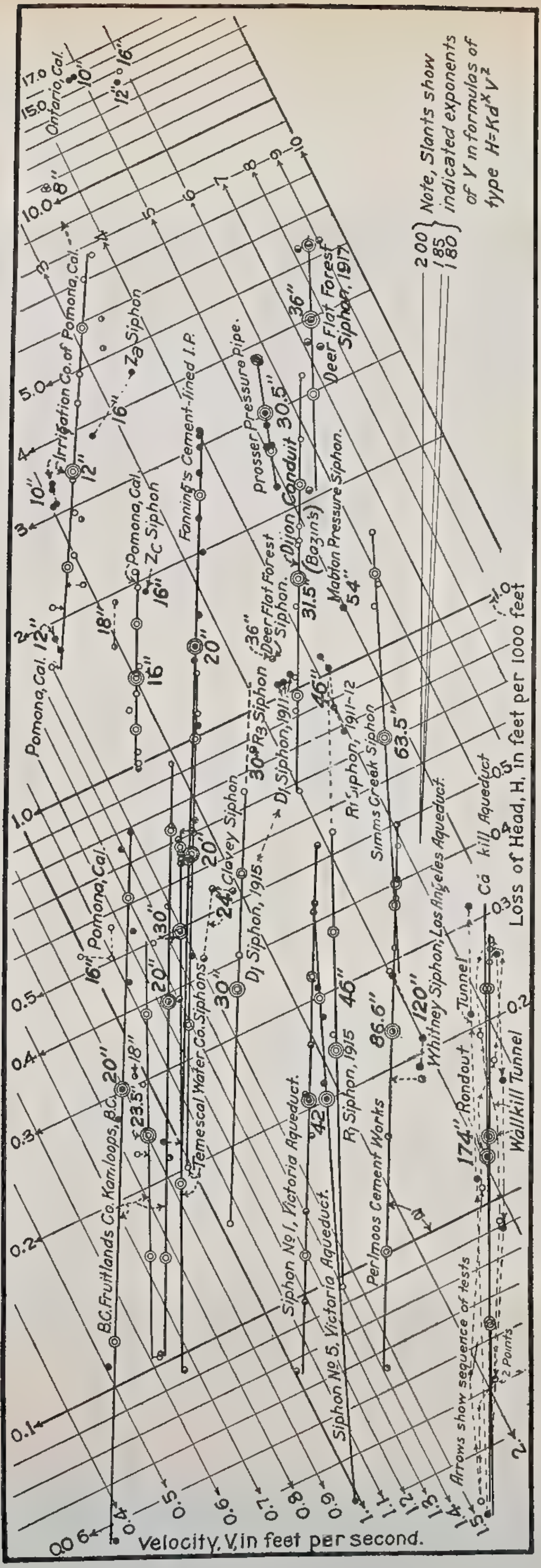




SHOWING OBSERVATIONS ON CONCRETE PIPES, RUNNING FULL.

This expressed logarithmically becomes

$$\log H = \log K + x \log d + z \log V \qquad (15)$$



LOGARITHMIC DIAGRAM SHOWING OBSERVATIONS ON CONCRETE PIPES, RUNNING FULL.

a low velocity in a large pipe is so small that it conflicts with the ordinary errors of experimentation.

For observation No. 1 (not included in Table 3) the water column at gauge No. 2 was 0.017 foot higher than the column at gauge No 1 for a velocity of 1.76 feet per second, whereas it should have been in the neighborhood of 0.070 lower. There was absolutely no error in the levels. Agreement between levels of the gauge glasses at the ends of the reach as developed by the wye level and as shown by the static pressure in the siphon with still water was within 0.001 foot. The writer can account for the discrepancy between the actual gauge heights (which indicated that there was a gain instead of a loss of head) and the heights that might be expected only by explaining that piezometer No. 1 is impinged upon by the water soon after pitching downhill at the intake of the siphon, while piezometer No. 2 is subjected to the current after the water has passed through 850 feet of 10-foot pipe, the last 200 feet of which are straight.

The observations on this pipe listed in Table 3 were made at velocities great enough so that a distinct loss of head was recorded, but there is probably some of the same error that showed the gain in head for observation No. 1. For this reason the writer does not accord any weight to this series, but does not wish to suppress the tests and uses them to emphasize the necessity of testing relatively long reaches of pipe in order that the actual loss of head may far overshadow the unavoidable experimental errors.

For experience on pipes of larger sizes see Appendix.

ANALYSIS OF THE EXPERIMENTAL DATA.

Water flowing under pressure, confined on all sides, probably follows a slightly different scheme as regards velocity distribution from that of water which but partially fills the conduit, thus having a surface exposed to the air. For this reason the results of experiments under these two conditions will be discussed separately.

FLOW IN PIPES UNDER PRESSURE.

It has come to be generally understood that the relationship of friction loss to velocity within a given pipe of any material can be represented by the equation

$$H = m\ V^z \tag{12}$$

in which the values of z are as a rule between 1.70 and 2.00, although there are many records of experiments in which the value of the exponent z was in excess of 2.

For a series of pipes of the same general characteristics but of varying diameters the values of m follow the general equation

$$m = K\ d^x \tag{13}$$

Substituting in formula 12

$$H = K\ d^x V^z \tag{14}$$

This expressed logarithmically becomes

$$\log H = \log\ K + x\ \log d + z\ \log\ V \tag{15}$$

In Plate VI the loss of head per thousand feet of pipe, H, for each observation on concrete pipes, is platted on logarithmic paper as an ordinate against the corresponding value of the mean velocity, V, as an abscissa. For a given series of observations the resulting curve represents equation 12 in its logarithmatic form

$$\log H = \log m + z \log V \tag{16}$$

which is now recognized as the equation of a straight line where m is the intercept on the axis of H (which is the line $V=1$) and z is the tangent of the angle between the curve and the axis of V (indicated by a on Pl. VI).

A study of this plate, in connection with the descriptions of the various pipes, shows that pipes of the same structural characteristics follow a rather definite order of position on the plot. If all of the curves were of the same inclination to the axis V, then this order of position would be definitely and fully disclosed by a diagram in which the values of m are platted on logarithmic paper as ordinates and the diameters of the pipes, in inches, are platted as abscissas. The experiments upon some of the pipes, of necessity, covered such a short range of velocities that the curves indicate a slope quite at variance with that which would have probably resulted for more complete series. For this reason the writer has not projected the curves to an intersection with the axis of H, in order to determine the relative positions of the values of m. Obviously some system of weighting should be assigned to the various series and, in the study of experiments upon wood-stave pipe, an arbitrary weighting method was employed; but because there was some criticism of this procedure the writer hesitates to repeat it.

If all the pipes were made in the same manner, had the same interior surfaces, and were subject to the same hydraulic conditions, a law derived by the method of least squares should be the best and most accurate. This method of handling experimental data, however, ascribes all variation from a given law to errors, according to the probability of errors, whereas most of the variation from an average law of data on commercially made concrete pipes is due to inherent differences in the pipes.

For any given series of experiments upon the same pipe the method of least squares is applicable, in its simplest form; that is, by the center-of-gravity method.[1] This method was used in computing the individual formulas given in column 10, Table 4. The curve is represented graphically in Plate VI, where the center of gravity of all the points in any one series is shown as two circles around a center which is typical of the observation points for that particular series. That is, if the observation points are given as open circles, then the center is an open circle, and if its points are given as solid dots, then the center is a solid dot. The centers of gravity of the upper and lower zones for a given series are shown by two concentric circles. The straight line representing the curve for that series passes through these three points, and the equation for this line is the equation for that particular pipe, so far as the observations developed it.

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[1] Described in Bul. 376, U. S. Dept. Agr., p. 50, and in Amer. Civil Engineers Pocketbook, 3d ed., New York, 1916, p. 847.

On Plate VI are shown three lines indicating slopes for three values of z. The slope of 1.80 conforms to that of the Moritz formula and to the slope found by both Moritz and the writer for the flow in wood-stave pipes. The slope of 1.85 conforms to that of the Williams-Hazen formula, while the slope of 2 conforms to the slope in the original Chezy formula and has been adhered to by later authorities. The border lines of the plate are also drawn at the slope of 2 for ready comparison. The slope of 2 agrees with the belief, so long honored that it became an axiom but was later proved not necessarily true, that the loss of head must vary as the square of the velocity.

With the desire not to increase the number of already numerous formulas, Plate VI was studied, on a tracing made over 10-inch logarithmic paper, in connection with sets of parallel lines based on the above-mentioned definite slopes. On this basis it was obvious that a slope of 2 most nearly applied, not only for the average slope of the various curves, but also in following the zone for a given size pipe from one series through a range of velocities to another series at higher or lower velocities.

Accepting this value for z of 2 and recognizing that there are several typical concrete surfaces, it is now necessary to determine x, the exponent of d and a set of coefficients, K. Allen Hazen has stated,[1] in discussing the Saph and Schoder experiments:

> It has seemed to the writer [Hazen] that the most accurate value for x could be secured by comparing the results obtained for very small and very large pipes. Of course it is impossible to secure very large pipes with precisely the same kind of interior surface as obtained in very small pipes, but it seems safer to compare the results obtained from very large and very small pipes, even though their interior surfaces do differ somewhat in character, than to take the indications of experiments more closely comparable, but covering a shorter range.

It is to be borne in mind that Hazen was speaking of brass pipes less than 2 inches in diameter when he referred to "very small pipes." However, the reasoning was sound, and this suggestion has been followed by the writer. Concrete pipes were divided into four general classes, as discussed on page 7: (1) Old California pipes; (2) modern "dry-mix" cement pipe and wood-form monolithic pipe; (3) wet-mix cement pipe and average steel-form monolithic pipe; (4)

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[1] Trans. Amer. Soc. Civil Engin., 51 (1903), p. 320.

glazed interior pipes. Class 3 was chosen from which to derive the value of x and of K_3, using the 20-inch Temescal pipe (No. 18), supported by Fanning's experiments (No. 19) for the small size and the Catskill aqueduct 174-inch pressure tunnels for the large size (Nos. 39 and 40). This class was chosen because the range between comparable surfaces was greater than for any of the other three classes, of which classes 1 and 2 extend from 6 inches up to about 36 inches, and class 4 is represented by experiments on pipes from 30 to 63 inches only.

The experiments upon the 20-inch and the 174-inch pipes mentioned above were accepted as basic, partially because they were conducted upon relatively long reaches of pipe where conditions for experimentation were favorable, but for the most part because of their relative positions on Plate VI, as regards the positions occupied by points representing pipes which are known to have very smooth surfaces and pipes that are known to be rougher than modern practice will countenance, as well as lines that can be classed as "modern concrete pipe," neither exceedingly smooth nor "mortar-squeeze" rough.

From the centers of gravity for the 20-inch and 174-inch pipes, projections at a slope of 2 intercept the axis of H (line for $V=1$) at 0.2006 and 0.0125 respectively, from which values of m for their respective diameters, K is computed as 9.4 and x as -1.28. Since x, the exponent of d, was accepted as -1.25 by Schoder and other authorities, and the writer did not desire to alter existing formulas unless the necessary change is radical, he also accepted -1.25 for x and recomputed K. The pivot for changing the slope from -1.28 to -1.25 should be for an average-size pipe rather than for a 1-inch pipe. A 42-inch pipe was accepted as about the average size; then substituting in formula 13,

$$m=\frac{9.4}{42^{1.28}},\ \text{or}\ \log m=\log 9.4-1.28\log 42$$

from which $m=0.07859$ for a diameter of 42 inches.

With this value of m and -1.25 for the value of x, substitution in formula 13 gives

$$0.07859=K\,42^{-1.25},\ \text{or}\ \log 0.07859+1.25\log 42=\log K.$$

or

$$\log K=10.92443,\ \text{from which}\ K=8.4.$$

Making the final basic formula for pipes of class 3 read:

$$H=\frac{8.4\ V^2}{d^{1.25}} \qquad (17)$$

from which

$$V=0.345\ H^{0.5}\ d^{0.625}=1.63\ H^{0.5}\ D^{0.625}=115\ R^{0.625}\ S^{0.5} \qquad (17a)$$

and since $Q=A\ V$, then

$$Q=0.00188\ H^{0.5}\ d^{2.625}=1.28\ H^{0.5}\ D^{2.625} \qquad (17b)$$

Class 1.—From a study of the older San Antonio Water Co. and Pomona pipes of class 1, a coefficient of 14 was found for K_1, making the formulas for this class read:

$$H=\frac{14\ V^2}{d^{1.25}} \tag{18}$$

from which

$$V=0.267\ H^{0.5}\ d^{0.625}=1.26\ \mathrm{H}^{0.5}\ D^{0.625}=89\ R^{0.625}\ S^{0.5} \tag{18a}$$

and

$$Q=0.00146\ H^{0.5}\ d^{2.625}=0.99\ H^{0.5}\ D^{2.625} \tag{18b}$$

Class 2.—Probably more than half the concrete pipes being laid at present in the United States are made locally, in 2-foot sections, by the "dry-mix" hand-tamped, cement-washed process in the west coast States. This type of pipe constitutes most of class 2. Throughout the irrigated portions of California nearly every town has one "pipe

For this class of pipes, a study of the newer Pomona and San Antonio Water Co. lines, supplemented by an examination of lines and pipe units in other parts of California, lead to the derivation of 10.4 for the coefficient K_2 making the formula for this class read

$$H=\frac{10.4\ V^2}{d^{1.25}} \tag{19}$$

from which

$$V=0.310\ H^{0.5}\ d^{0.625}=1.47\ H^{0.5}\ D^{0.625}=103.5\ R^{0.625}\ S^{0.5} \tag{19a}$$

and

$$Q=0.00169\ H^{0.5}\ d^{2.625}=1.15\ H^{0.5}\ D^{2.625} \tag{19b}$$

Class 4.—The experiments upon the Victoria Aqueduct, siphon No. 1 (pipe No. 30) were chosen as characteristic, because they are well supported on the conservative side by those on other exceedingly smooth finished pipes and the conditions for experimentation were so unusually favorable. The coefficient K_4 was found to be 6.7, making the formula for this class read

$$H=\frac{6.7\ V^2}{d^{1.25}} \tag{20}$$

from which

$$V=0.370\ H^{0.5}\ d^{0.625}=1.75\ H^{0.5}\ D^{0.625}=123.5\ R^{0.625}\ S^{0.5} \tag{20a}$$

and

$$Q=0.00202\ H^{0.5}\ d^{2.625}=1.37\ H^{0.5}\ D^{2.625} \tag{20b}$$

EFFECT OF AGE UPON CARRYING CAPACITY.

In designing a pipe line of a given material and workmanship the engineer must not consider so much the capacity of the pipe when new but after a period of subjection to the local conditions that finally determine the interior of any pipe line. Deposits of mud, gravel, etc., will of course choke any pipe and will not be considered in the following discussion, though it may be well to state in passing that accumulations of detritus do not occur at the low point in the profile of a siphon pipe, but will pass this point and precipitate at the base of the first steep upward pitch. They are thus, as a rule, beyond the reach of a flushing stream if the only blow-off is located at the extreme low point. For this reason it is advisable to place the main blow-off at the steep upward pitch and a small valve at the extreme low point in order that the pipe may be completely drained.

It has become an accepted fact that cast-iron and steel pipes carry less water for a given loss of head from year to year. So far as evidence is obtainable, wood-pipe interiors remain about the same, though occasionally a pipe is found with interior growths of *Spongilla.*

Moritz writes:

> The effect of age on concrete pipe is not known, but it is customary to assume that the carrying capacity does not decrease, as there is no reason to suppose that it should.[1]

Bishop states that there appears to be no deterioration of capacity in cement-lined pipes laid in 1871–72 for municipal purposes. His examination was made 38 years later.[2]

The city authorities of Grenoble, France, comment favorably upon the condition of a concrete pipe after 15 years' service, mentioning a slight calcareous deposit one-sixteenth inch thick.[3]

At first thought it would appear that cement or concrete, subjected to water free from erosive material, should retain the original surface, with the possible exception of sliming, which may cut down the capacity from 10 to 15 per cent, but from which concrete surfaces are comparatively free.[4]

Examination of the interiors of many pipe lines supplemented by interviews with managers of water systems and a study of available literature show that the capacity may deteriorate, due to the following causes:

(1) The invasion of rootlets from neighboring trees.
(2) Scour from the water.
(3) The accumulation of lime or other deposits.

In southern California experience teaches that pipe lines may be laid under citrus groves with no ill effects from roots, but that where the line is laid near black acacia, willow, pepper, or eucalyptus trees, rootlets will enter the pipe, not necessarily at the joints, if it is poorly made and poorly laid, and retard the flow. These trees are named in order, the acacia being the most destructive. Roots will not effect a dense, well-made pipe, with tight joints. (See Mr. Finkle's discussion on p. 95.)

Literature contains accounts of clay and concrete pipes being clogged with roots, particularly of the elm tree.[5]

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[1] Working Data for Irrigation Engineers, p. 69.

[2] Experience with Cement Mains at Rahway, N. J. By Wm. Bishop, Pro. American Waterworks Assn. (1910), p. 217.

[3] Can. Engr., Oct. 19, 1911, vol. 21, p. 455.

[4] Water Works Handbook, by Flinn, Weston, and Bogert, New York, 1916, p. 290.

[5] Eng. Rec., 1916, vol. 73, p. 460; *id.*, p. 514; *id.*, p. 688; *id.*, vol. 79, p. 36. Irrigation in Southern California, W. H. Hall, Sacramento, Calif., 1888, p. 495.

Concrete, under usual conditions, will withstand high velocities.[1] According to A. P. Davis, the following conclusions were warranted from his observation of velocities in concrete:

> 1. That where clear water can be made to glide over concrete without disturbing its velocity or abruptly changing its direction, there is no practical limit to the velocities that can be permitted without harm.
> 2. That concrete which is subjected to the impact of water under high velocity is rapidly eroded and that under such conditions the velocities must be very carefully limited.

The fact has been pointed out by C. H. Paul that concrete tunnels treated with a coat of water-gas tar, followed with two coats of coal tar, withstood velocities up to 64 feet per second without appreciable wear.[2] The experience with this feature of water flow, gained in making these tests, may be of value. The water conveyed through the Prosser pressure pipe (No. 26) contains many fine particles of hard, rough, basalt ravelings. This pipe is operated most of the time at much less than its maximum capacity, so that the water enters the intake in a very turbulent condition and rushes down the initial reaches of the pipe at a high velocity. At the intake the bottom of the pipe presents the appearance of having been subjected to a sand blast. All the finer materials in the concrete have been scoured out, clearly defining each hard pebble larger than about one-fourth inch in diameter. This scour has extended possibly one-eighth inch deep between pebbles. The degree of roughness diminishes from a maximum at the bottom to none at the mid-diameter; likewise diminishing with distance down the pipe until it was not noticeable about 150 feet from the intake. (See Mr. Newell's discussion on p. 100.)

A peculiar condition under which erosion may be expected was called to the attention of the writer in southern California. Unless pipe lines are laid on a smooth gradient, any sand in the water will wear out the bottom of the pipe where the latter goes over humps.

The obstruction of a pipe by lime deposit is mentioned by W. E. Condon.[3] (See Plate I, figure 2.)

CAPACITY OF CONCRETE PIPES.

In the following pages the design of concrete pipes is considered with reference to carrying capacity alone. Structural features do not come within the scope of this paper, except as they affect the interior surface.

The total loss of head necessary in the conveyance of a given quantity of water will be the sum of the velocity head, h_v, the entry head, h_e, and the friction head, h_f, or its equivalent per unit length, less any velocity head, h'_v, that may be recovered as the water approaches the pipe outlet at a velocity relatively high compared with the velocity in the open water below the outlet chamber. This total may be expressed by the formula

---

[1] Safe Velocities of Water in Concrete, by A. P. Davis, Eng. News, vol. 67, Jan. 4, 1912, p. 20. Engin. News-Rec., vol. 80, p. 172. Concrete, Plain and Reinforced, by Taylor and Thompson, New York, 1917, p. 779.

[2] Use of Water-Gas and Coal Tar on Concrete Subjected to High Velocities of Water, by C. H. Paul, Reclamation Record, Jan., 1916, p. 46. Reprinted in Engin. Rec., Jan. 22, 1916, p. 108.

[3] Original and Acquired Roughness of a 30-inch Cement Water Supply Conduit, Southern California, Engin. News, Jan. 9, 1908, vol. 159, p. 41.

$$H_e = h_v + h_e + h_f - h'_v \tag{20}$$

where H_e has the significance shown in figure 1 and h_v, h_e, and h_f, and h_v' have the significance defined on page 51. The influence of gentle curves was included in the data upon which the formulas were based, so that an additional loss for slight bends or curves need not be considered in the design of the usual pipe on irrigation systems. If sharp bends can not be avoided, then an additional loss of head must be anticipated. The results of such tests as have been made on bends in pipes are given in standard works on hydraulics.

VELOCITY AND ENTRY LOSSES.

In designing pipes of small diameter and great length, the losses due to velocity and entry heads, h_v and h_e, are so small compared with the friction loss that they may be neglected. Otherwise they should be included in the allowance for total lost head.

It is best to consider the water above any intake as at rest. From this state of rest velocity must be created and increased to the mean velocity, V, existing in the pipe. The head, h_v, necessary to create a given velocity is shown in column 2, Table 5. The entry loss will be from 0.5 h_v where the pipe of standard size begins at a headwall and is without bell or taper mouth, to about 0.25 h_v, for a rounded intake, and 0.05 h_v for a bell-mouth intake. In Table 5, column 3, is shown the amount of entry loss when taken as half the velocity head (column 2), and the sum of the entry and velocity losses is shown in column 4.

TABLE 5.—*Mean velocity in pipe V, in feet per second, and head of elevation lost creating this velocity and overcoming entrance conditions, h_v and h_e, in feet.*

1	2	3	4	1	2	3	4
V	h_v	h_e	h_v+h_e	V	h_v	h_e	h_v+h_e
Ft. per sec.	*Feet.*	*Feet.*	*Feet.*	*Ft. per sec.*	*Feet.*	*Feet.*	*Feet.*
1.0	0.016	0.008	0.024	5.0	0.389	0.195	0.584
.2	.022	.011	.033	.2	.420	.210	.630
.4	.030	.015	.045	.4	.453	.227	.680
.6	.040	.020	.060	.6	.488	.244	.732
.8	.050	.025	.075	.8	.523	.262	.785
2.0	.062	.031	.093	6.0	.560	.280	.840
.2	.075	.037	.112	.2	.598	.299	.897
.4	.090	.045	.135	.4	.637	.319	.956
.6	.105	.053	.158	.6	.677	.339	1.016
.8	.122	.061	.183	.8	.719	.359	1.078
3.0	.140	.070	.210	7.0	.762	.381	1.143
.2	.159	.080	.239	.2	.806	.403	1.209
.4	.180	.090	.270	.4	.851	.426	1.277
.6	.202	.101	.303	.6	.898	.449	1.347
.8	.224	.112	.336	.8	.946	.473	1.419
4.0	.249	.125	.374	8.0	.995	.498	1.493
.2	.274	.137	.411	.2	1.045	.523	1.568
.4	.301	.151	.452	.4	1.097	.549	1.646
.6	.329	.165	.494	.6	1.150	.575	1.725
.8	.358	.179	.537	.8	1.204	.602	1.806

Where the usual types of inlet and outlet structure are employed, with but little construction and consequent expense incurred for conservation of entry and velocity heads, it is recommended that the figures in Table 5 be used.

AIR IN PIPE.

Like all other lines, a concrete pipe must be protected by air valves against the accumulation of air at "summits" on the line. If this is not done the capacity will be reduced.

In addition to the air at summits it has been found that the capacity of a siphon pipe may be reduced by accumulations of air at the intake end. This accumulation may take place several hundred feet down the incline from the pipe entrance and manifest itself by a periodic "blowing back" of compressed air. The writer has seen two instances where the resulting conflict between air and water has burst the pipe.

Pipes taking water directly from reservoirs are, of course, not subject to these troubles, the depth above the intake being, as a rule, sufficient to insure filling of the pipe with water alone.

Mr. A. E. Ashcroft, of Vernon, British Columbia, Canada, in speaking of some 36-inch siphon pipes, says:[1]

> A large number of air valves at the upper ends of the pipes have been effective in reducing vibration and thumping when the pipes are discharging only partially full. (See also Mr. Finkle's discussion, p. 94.)

The best air outlet is probably a "chimney" rising above the hydraulic gradient. These vents may be from 1 to 36 inches in diameter, depending on the size of the pipe line, and small ones should be so assembled that they may be taken apart, as débris collects in such vents and must be periodically removed. Moritz suggests the area of the relief pipe be one-twentieth of that of the pressure pipe.[2]

FRICTION LOSSES.

The loss of head necessary to overcome internal resistances within the pipe is proportional to the length of the pipe, but is independent of the static pressure in the pipe.

In order to determine the size of pipe and the loss of head necessary to overcome the frictional resistances in the conveyance of a given quantity of water, three estimate diagrams and four tables have been prepared. Examples of typical pipe problems are given. The factors of safety given below should be considered in each problem.

FACTORS OF SAFETY.

As in all conservative design or estimating, factors of safety must be used in order to take care of the divergence, due to unforeseen or abnormal conditions, from a general average.

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[1] Pro. 4th An. Conv., W. Can. Irg. Assoc., Ottawa, 1911, p. 76.

[2] Working Data for Irrigation Engineers, p. 70.

In general a lower factor of safety may be used for a jointed pipe of precast units than for a pipe of monolithic construction. This is true because a given surface may be more closely anticipated before construction.

For pipes of classes 1 and 4 (p. 7) the velocity may be assumed to be within 10 per cent of that computed by the new formula, provided the conditions are favorable.

For pipes of classes 2 and 3 the writer would suggest about the same factors of safety as for wood-stave pipe:

> Five per cent when only a rough approximation to the actual needs of the pipe is possible; when water enters the pipe from a settling reservoir or velocities in the pipe are so high that a clean-scoured condition will always be present inside the pipe; and also where conditions of operation are such that no penalties are attached to a slight insufficiency of carrying capacity.
>
> Ten per cent when the above conditions for a very clean pipe are assured, but where penalties are attached to lack of capacity; or where no direct penalties are attached but silted waters and low velocities may permit deposits.
>
> Fifteen per cent where rock ravelings may reduce the interior area of the pipe, or when penalties are attached and silted water is likely to cause excess retardation of flow, or where chemical analyses of the water indicate that accretions may be expected.
>
> The designer may safely assume that the capacity of concrete will not change materially unless the pipe is subject to conditions mentioned on pages 50 and 51.

As a factor for safety to be used in the design of a pipe line, the writer would suggest adding the percentage to the load to be carried rather than a change of coefficients—that is, if 100 second-feet of water is the desired quantity and a factor of 10 per cent is to be used, then design the line for a capacity of 110 second-feet.

ESTIMATE DIAGRAMS AND TABLE; SOLUTIONS FOR TYPICAL PIPE PROBLEMS.

(1) An inverted siphon of class 3 is required to convey 60 second-feet of water a length of 2,800 feet, the velocity at peak load to be about 5 feet per second. Water has settled in a reservoir before entering the canal. The siphon is to contain no abrupt turns or obstructions. No direct penalty has been attached for lack of capacity. Required, diameter of pipe.

Allowing a 5 per cent overload as a factor of safety, the rated capacity will be $60+3=63$ second-feet. At a velocity of 5 feet per second the area of the water cross section—that is, the inside area of the pipe—must be $\frac{63}{5}=12.6$ square feet, which is close to the area of a 48-inch pipe. On Plate VII, from the intersection of diameter line 48 and coefficient line 0.345, follow the guide lines to an intersection with velocity line 5. This intersection is on loss-of-head line 1.7 feet per thousand feet of pipe. Thus for a 4-foot pipe 2,800 feet long the friction loss incident to the conveyance of 60 second-feet

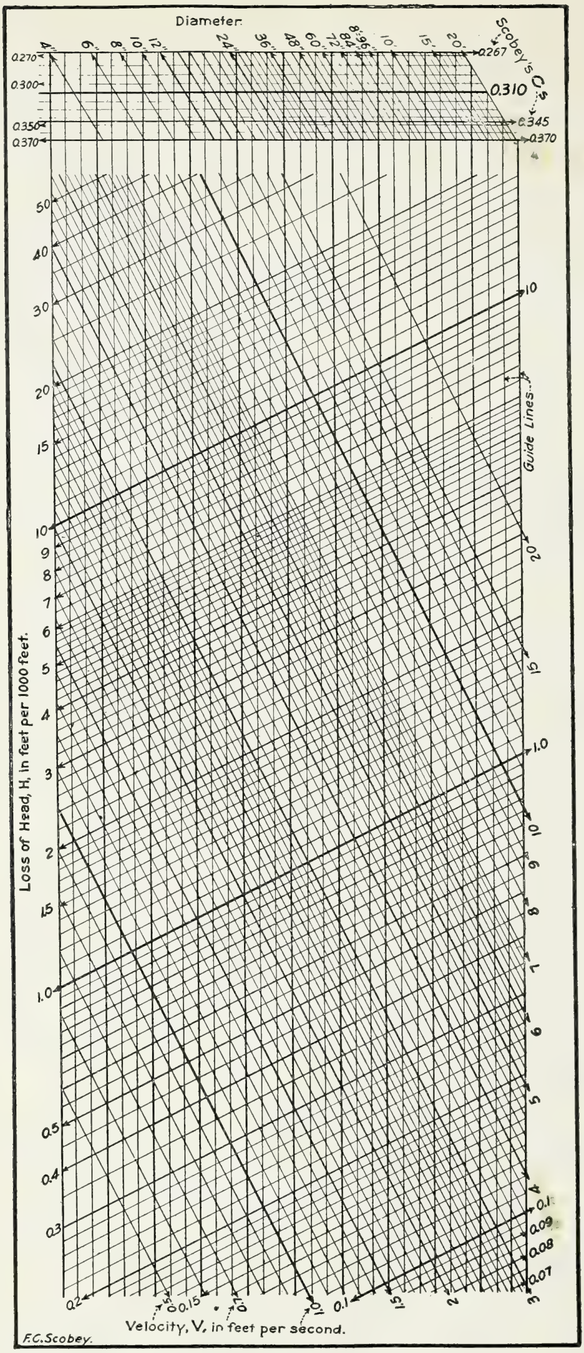

DIAGRAM FOR USE IN THE DESIGN OF CONCRETE PIPE LINES.

From the intersection of the diameter and C_s follow guide lines to the intersection of H and V, or from the intersection of H and V follow guide lines to the intersection of diameter and C_s. No straight edge required. For recommended values of C_s, see page 7.

will be $1.7 \times 2.8 = 4.76$ feet. In addition to this the loss of head necessary to create the velocity of 5 feet per second is $\frac{5^2}{64.4} = 0.4$ foot. Assuming the entry head as one-half the velocity head gives an additional loss of 0.2 foot. Since for most installations there is but little or no recovery of velocity head at the outlet, it may be assumed that the total loss of head will approximate $4.8 + 0.4 + 0.2 = 5.4$ feet. If it is not feasible to sacrifice 5.4 feet a lower maximum velocity may be assumed and the above process repeated, this of course resulting in a larger pipe. If elevation is of little moment, that is, if more grade may be sacrificed, then a higher velocity may be assumed and a smaller pipe used.

The same results may be obtained by interpolating in Table 8.

(2) An orchardist wishes to convey the output of a 5-inch centrifugal pump, rated at 700 gallons per minute, from a standpipe near the well to the high corner of his orchard which is 5 feet higher than the land at the standpipe, and 1,370 feet distant along the line the pipe must follow. The owner wishes to use 12-inch pipe made by the dry-mix process. How high must the standpipe be to deliver the water at the high corner of the orchard with a pressure head of 2 feet still available to insure a free discharge? To provide for an assured capacity of the line, for a slight undersize in the pipe, and a slight overload for the pump, we will figure on 15 per cent more than 700 gallons per minute or $700 + 105 = 805$ gallons per minute. In column 4, Table 6, we find the item 809 as nearest to 805. This is close enough for our computations. Opposite 809 in the columns under a 12-inch pipe we find the velocity V, of such a flow in a 12-inch pipe will be 2.29 feet per second and the friction head for 1,000 feet of pipe, H, will be 2.44 feet. Since our pipe will be 1,370 feet long, this friction head will be $1.37 \times 2.44 = 3.34$ feet. A small amount of fall is also necessary in order to generate the velocity of 2.29 feet per second and to get the water from the standpipe into the flow line. In Table 5, page 52, we find that a velocity of 2.4 feet per second requires a head (fall) of 0.135 foot, according to column 4. Thus the following items enter into the total height of our standpipe:

	Feet.
Difference in elevation of the ground surface	5. 00
Reserve pressure head at outlet	2. 00
Friction loss in 1,370 feet of 12-inch pipe for 809 G. P. M.	3. 34
Head to generate velocity and get water into the pipe	. 13
Total	10. 47

So a standpipe 11½ or 12 feet high, measured from the ground surface, will be sufficient to insure the flow and have some elevation in reserve for "freeboard" and surging.

TABLE 6.—FOR ESTIMATING THE CAPACITY OF MODERN CONCRETE PIPE LINES MADE BY DRY-MIXTURE PROCESS.[1]

Based on formula $V = C_s H^{0.5} d^{0.625}$ using 0.310 for the coefficient C_s. For instance, 2 second-feet of water, the equivalent of 80 miner's inches measured under a 6-inch pressure, or of 100 miner's inches measured under a 4-inch pressure, or of 898 gallons per minute, will be conveyed by a 12-inch pipe, with a velocity of 2.55 feet per second with a loss of head (grade) of 3.04 feet for each 1,000 feet of pipe.

Quantity (Q).				Inside diameter, in inches and corresponding area, A, in square feet.											
Sec.-ft.	Miner's inches.		Gallons per minute.	6 A=0.196		8 A=0.349		10 A=0.545		12 A=0.785		14 A=1.069		16 A=1.396	
	A(40)	B(50)		*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*
				Feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
0.1	4	5	45	0.51	0.28	0.29	0.07	......	......	......	......	......	......	......	......
.2	8	10	90	1.02	1.06	.57	.15	......	......	......	......	......	......	......	......
.3	12	15	135	1.53	2.60	.86	.57	......	......	......	......	......	......	......	......
.4	16	20	180	2.04	4.61	1.14	1.00	......	......	......	......	......	......	......	......
.5	20	25	225	2.55	7.24	1.43	1.58	......	......	......	......	......	......	......	......
.6	24	30	270	3.06	10.40	1.72	2.30	1.10	0.71	0.76	0.27	......	......	......	......
.7	28	35	315	3.56	14.00	2.01	3.15	1.28	.96	.89	.37	......	......	......	......
.8	32	40	360	4.08	18.40	2.29	4.07	1.47	1.26	1.02	.48	0.75	0.22	......	......
.9	36	45	405	4.58	23.40	2.58	5.16	1.65	1.60	1.14	.61	.84	.27	......	......
1.0	40	50	449	5.10	28.80	2.87	6.38	1.83	1.96	1.27	.75	.94	.34	......	......
1.2	48	60	539	6.11	42.00	3.44	9.15	2.20	2.84	1.53	1.09	1.12	.48	0.86	0.24
1.4	56	70	629	7.13	56.00	4.01	12.50	2.57	3.88	1.78	1.48	1.31	.66	1.00	.33
1.6	64	80	719	8.15	74.00	4.58	16.30	2.93	5.07	2.04	1.95	1.50	.84	1.15	.43
1.8	72	90	809	9.17	93.00	5.16	20.70	3.30	6.46	2.29	2.44	1.68	1.09	1.29	.54
2.0	80	100	898	10.20	105.00	5.73	25.40	3.67	7.95	2.55	3.04	1.87	1.35	1.43	.67
2.2	88	110	988	11.20	140.00	6.30	30.80	4.03	9.50	2.80	3.66	2.06	1.63	1.58	.82
2.4	96	120	1,089	12.20	165.00	6.87	36.50	4.40	11.40	3.06	4.37	2.24	1.93	1.72	.97
2.6	104	130	1,168	......	......	7.45	43.00	4.77	13.30	3.31	5.11	2.43	2.28	1.86	1.13
2.8	112	140	1,258	......	......	8.02	50.00	5.13	15.50	3.56	5.92	2.62	2.64	2.00	1.30
3.0	120	150	1,348	......	......	8.59	57.30	5.50	17.80	3.82	6.81	2.81	3.04	2.15	1.50
3.2	128	160	1,438	......	......	9.17	65.30	5.87	20.20	4.07	7.74	2.99	3.44	2.29	1.71
3.4	136	170	1,528	......	......	9.74	73.50	6.23	22.80	4.33	8.77	3.18	3.90	2.44	1.94
3.6	144	180	1,618	......	......	10.30	82.50	6.60	25.60	4.58	9.82	3.37	4.37	2.59	2.18
3.8	152	190	1,708	......	......	10.90	92.20	6.97	28.50	4.84	10.80	3.55	4.86	2.72	2.41
4.0	160	200	1,797	......	......	......	......	7.33	31.50	5.09	12.20	3.74	5.39	2.86	2.67
4.5	180	225	2,022	......	......	......	......	8.25	39.70	5.73	15.30	4.21	6.81	3.24	3.64
5.0	200	250	2,247	......	......	......	......	9.17	49.10	6.37	18.80	4.68	8.41	3.58	4.53
5.5	220	275	2,472	......	......	......	......	10.10	59.60	7.00	22.80	5.15	10.20	3.94	5.49
6.0	240	300	2,697	......	......	......	......	11.00	70.70	7.64	27.10	5.61	12.10	4.30	6.54
6.5	260	325	2,922	......	......	......	......	11.90	82.70	8.27	31.80	6.08	14.20	4.66	7.67

Sec.-ft.	Miner's inches.		Gallons per minute.	18 A=1.767		20 A=2.182		22 A=2.640		24 A=3.142		30 A=4.909		36 A=7.068	
	A(40)	B(50)		*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*
4.5	180	225	2,022	2.55	1.85	2.06	1.05	1.70	0.63	1.43	0.40	0.92	0.13	0.64	0.05
5.0	200	250	2,247	2.83	2.25	2.29	1.29	1.89	.78	1.59	.50	1.02	.16	.71	.06
5.5	220	275	2,472	3.11	2.72	2.52	1.57	2.08	.95	1.75	.60	1.12	.19	.78	.07
6.0	240	300	2,697	3.40	3.24	2.75	1.86	2.27	1.13	1.91	.71	1.22	.22	.85	.09
6.5	260	325	2,922	3.68	3.80	2.98	2.18	2.46	1.32	2.07	.84	1.32	.26	.92	.10
7.0	280	350	3,146	3.96	4.40	3.21	2.54	2.65	1.57	2.23	.97	1.42	.30	.99	.12
7.5	300	375	3,371	4.24	5.05	3.44	2.92	2.84	1.77	2.39	1.12	1.53	.35	1.06	.13
8.0	320	400	3,596	4.53	5.77	3.64	3.32	3.03	2.01	2.55	1.28	1.63	.39	1.13	.15
8.5	340	425	3,821	4.81	6.50	3.90	3.74	3.22	2.26	2.71	1.44	1.73	.45	1.20	.17
9.0	360	450	4,046	5.10	7.30	4.12	4.18	3.41	2.54	2.86	1.50	1.83	.50	1.27	.19
9.5	380	475	4,271	5.38	8.16	4.35	4.66	3.60	2.84	3.02	1.79	1.94	.56	1.34	.21
10.0	400	500	4,496	5.66	8.98	4.58	5.17	3.79	3.15	3.18	1.98	2.04	.62	1.41	.24
11.0	440	550	4,948	6.22	10.80	5.04	6.26	4.17	3.80	3.50	2.41	2.24	.74	1.56	.29
12.0	480	600	5,398	6.79	12.90	5.50	7.43	4.55	4.53	3.82	2.86	2.45	.88	1.70	.34
13.0	520	650	5,847	7.36	15.20	5.96	8.76	4.92	5.29	4.14	3.37	2.65	1.04	1.84	.40
14.0	560	700	6,297	7.92	17.70	6.42	10.20	5.30	6.14	4.46	3.90	2.85	1.20	1.98	.46
15.0	600	750	6,747	8.49	20.30	6.87	11.70	5.60	6.85	4.77	4.46	3.06	1.39	2.12	.53
16.0	640	800	7,197	9.06	23.00	7.33	13.20	6.06	8.02	5.09	5.08	3.26	1.57	2.26	.60
17.0	680	850	7,647	9.62	26.00	7.79	14.90	6.44	9.05	5.42	5.75	3.46	1.77	2.40	.68
18.0	720	900	8,096	10.20	29.30	8.25	16.80	6.82	10.20	5.73	6.45	3.67	2.00	2.55	.77
20.0	800	1,000	8,996	11.30	35.90	9.17	20.70	7.58	12.60	6.37	7.94	4.07	2.46	2.83	.94
22.0	880	1,100	9,896	12.40	43.30	10.10	25.10	8.33	15.10	7.00	9.58	4.48	2.98	3.11	1.14
24.0	960	1,200	10,795	13.60	51.90	11.00	28.80	9.09	18.10	7.64	11.40	4.89	3.55	3.40	1.37
26.0	1,040	1,300	11,695	14.70	60.80	11.90	34.90	9.85	21.30	8.27	13.40	5.30	4.10	3.68	1.60
28.0	1,120	1,400	12,594	15.80	70.10	12.80	40.40	10.60	24.60	8.91	15.60	5.70	4.82	3.96	1.85
30.0	1,200	1,500	13,492	......	......	13.70	46.20	11.40	28.40	9.55	17.90	6.11	5.59	4.24	2.12
32.0	1,280	1,600	14,390	......	......	14.70	53.30	12.10	32.00	10.20	20.40	6.52	6.30	4.53	2.43

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[1] This is the table that should be used in the design of pipe lines made from units 2 to 3 feet long, as ordinarily made by hand in the pipe yards of the Pacific Coast States.

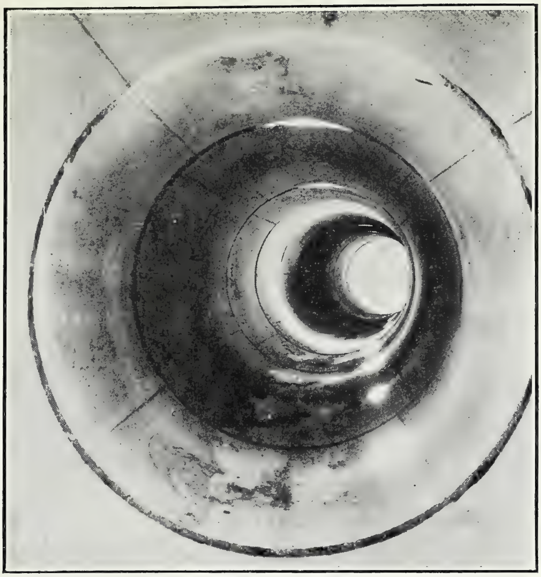

FIG. I.—FLOW LINE ON VICTORIA AQUEDUCT.

Interior photographed before grouting joints. Spots of sunlight come through "soup holes" in top of pipe line.

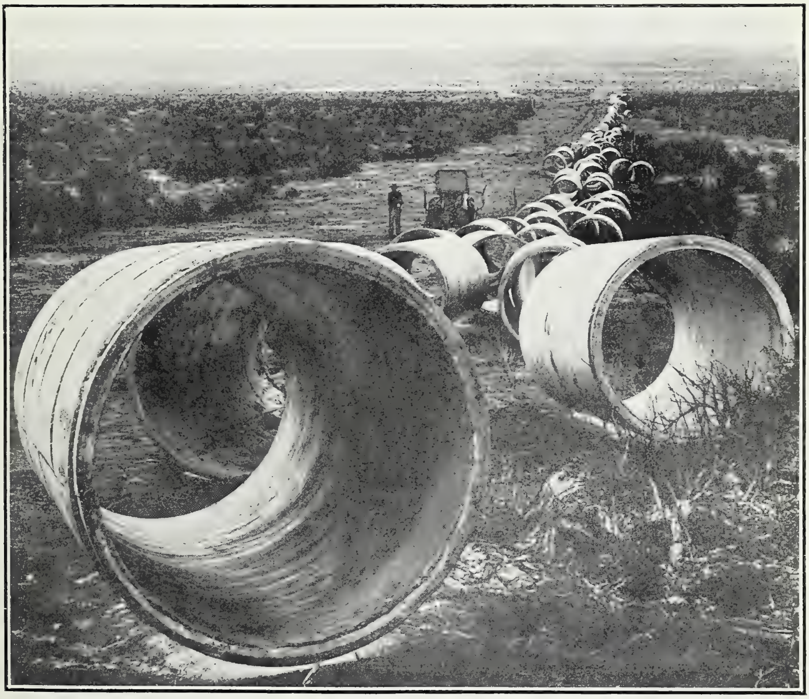

FIG. 2.—R1 LINE, UMATILLA PROJECT, U. S. RECLAMATION SERVICE, OREGON. (PIPES NOS. 32 AND 33.)

Inlet at end of clearing in distance. This line under 110 feet of pressure head.

TABLE 7.—FOR ESTIMATING THE CAPACITY OF MODERN "WET-MIX" JOINTED AND WELL-MADE MONOLITHIC CONCRETE PIPE LINES.

Based on formula $V = C_s H^{0.5} d^{0.625}$ using 0.345 for the coefficient C_s. For instance, 3 second-feet of water, the equivalent of 120 miner's inches of type A or 150 inches of type B, will be conveyed by a 16-inch pipe at a velocity of 2.15 feet per second with a loss of head (grade) of 1.22 feet per thousand feet of pipe.

Quantity (Q).			Inside diameter in inches and corresponding area, A, in square feet.											
			6 $A=0.1963$		**8** $A=0.3491$		**10** $A=0.5454$		**12** $A=0.7854$		**14** $A=1.069$		**16** $A=1.396$	
	$A(40)$[1]	$B(50)$[1]	V	H	V	H	V	H	V	H	V	H	V	H
			Feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
0.1	4	5	0.51	0.23	0.29	0.05	......	......	......	......	......	......	......	......
.2	8	10	1.02	.93	.57	.20	......	......	......	......	......	......	......	......
.3	12	15	1.53	2.09	.86	.46	......	......	......	......	......	......	......	......
.4	16	20	2.04	3.72	1.14	.81	0.73	0.25	......	......	......	......	......	......
.5	20	25	2.55	5.82	1.43	1.28	.92	.40	......	......	......	......	......	......
.6	24	30	3.06	8.38	1.72	1.85	1.10	.57	0.76	0.22	......	......	......	......
.7	28	35	3.56	11.30	2.01	2.52	1.28	.77	.89	.30	......	......	......	......
.8	32	40	4.08	14.90	2.29	3.28	1.47	1.02	1.02	.39	0.75	0.18	......	......
.9	36	45	4.58	18.80	2.58	4.16	1.65	1.28	1.14	.49	.84	.22	......	......
1.0	40	50	5.09	23.20	2.87	5.14	1.83	1.58	1.27	.61	.94	.27	0.72	0.14
1.2	48	60	6.11	33.40	3.44	7.39	2.20	2.29	1.53	.88	1.12	.39	.86	.19
1.4	56	70	7.13	45.50	4.01	10.00	2.57	3.12	1.78	1.19	1.31	.53	1.00	.26
1.6	64	80	8.15	59.40	4.58	13.10	2.93	4.06	2.04	1.56	1.50	.70	1.15	.35
1.8	72	90	9.17	75.20	5.16	16.60	3.30	5.14	2.29	1.97	1.68	.88	1.29	.44
2.0	80	100	10.20	93.00	5.73	20.50	3.67	6.36	2.55	2.46	1.87	1.09	1.43	.54
2.2	88	110	11.20	112.00	6.30	24.80	4.03	7.67	2.80	2.95	2.06	1.32	1.58	.66
2.4	96	120	12.20	133.00	6.87	29.50	4.40	9.14	3.06	3.52	2.24	1.56	1.72	.78
2.6	104	130	13.20	156.00	7.45	34.70	4.77	10.80	3.31	4.12	2.43	1.83	1.86	.91
2.8	112	140	14.30	182.00	8.02	40.20	5.13	12.40	3.56	4.76	2.62	2.13	2.00	1.05
3.0	120	150	15.30	209.00	8.59	46.10	5.50	14.30	3.82	5.49	2.81	2.45	2.15	1.22
3.2	128	160	......	......	9.17	52.50	5.87	16.30	4.07	6.23	2.99	2.77	2.29	1.37
3.4	136	170	......	......	9.74	59.30	6.23	18.30	4.33	7.06	3.18	3.14	2.44	1.56
3.6	144	180	......	......	10.30	66.30	6.60	20.60	4.58	7.89	3.37	3.52	2.59	1.76
3.8	152	190	......	......	10.90	74.20	6.97	23.00	4.84	8.81	3.55	3.91	2.72	1.94
4.0	160	200	......	......	11.50	82.60	7.33	25.40	5.09	9.75	3.74	4.34	2.86	2.15
			18 $A=1.767$		**20** $A=2.182$		**22** $A=2.640$		**24** $A=3.142$		**30** $A=4.909$		**36** $A=7.068$	
			V	H	V	H	V	H	V	H	V	H	V	H
2.0	86	100	1.13	0.29	0.92	0.17	0.76	0.10	0.64	0.06	......	......	......	......
2.5	100	125	1.42	.46	1.15	.26	.95	.16	.80	.10	0.51	0.03	......	......
3.0	120	150	1.70	.65	1.37	.37	1.14	.23	.95	.14	.61	.04	......	......
3.5	140	175	1.98	.89	1.60	.51	1.33	.31	1.11	.20	.71	.06	0.50	0.02
4.0	160	200	2.23	1.13	1.83	.66	1.52	.41	1.27	.26	.81	.08	.57	.03
4.5	180	225	2.55	1.47	2.06	.84	1.70	.51	1.43	.32	.92	.10	.64	.04
5.0	200	250	2.83	1.81	2.29	1.04	1.89	.63	1.59	.40	1.02	.12	.71	.05
5.5	220	275	3.11	2.19	2.52	1.26	2.08	.76	1.75	.48	1.12	.15	.78	.06
6.0	240	300	3.40	2.62	2.75	1.50	2.27	.91	1.91	.58	1.22	.19	.85	.07
6.5	260	325	3.68	3.07	2.98	1.76	2.46	1.07	2.07	.68	1.32	.21	.92	.08
7.0	280	350	3.96	3.55	3.21	2.05	2.65	1.24	2.23	.79	1.42	.24	.99	.09
7.5	300	375	4.24	4.07	3.44	2.35	2.84	1.42	2.39	.90	1.53	.28	1.06	.11
8.0	320	400	4.53	4.65	3.67	2.68	3.03	1.62	2.55	1.03	1.63	.32	1.13	.12
8.5	340	425	4.81	5.24	3.90	3.02	3.22	1.83	2.71	1.17	1.73	.36	1.20	.14
9.0	360	450	5.10	5.90	4.12	3.37	3.41	2.05	2.86	1.29	1.83	.40	1.27	.15
9.5	380	475	5.38	6.56	4.35	3.76	3.60	2.29	3.02	1.44	1.94	.45	1.34	.17
10	400	500	5.66	7.26	4.58	4.17	3.79	2.53	3.18	1.60	2.04	.50	1.41	.19
11	440	550	6.22	8.77	5.04	5.04	4.17	3.07	3.50	1.94	2.24	.60	1.56	.23
12	480	600	6.79	10.40	5.50	6.01	4.55	3.65	3.82	2.31	2.44	.71	1.70	.28
13	520	650	7.36	12.30	5.96	7.06	4.92	4.27	4.14	2.71	2.65	.84	1.84	.32
14	560	700	7.92	14.20	6.42	8.19	5.30	4.95	4.46	3.15	2.85	.97	1.98	.37
15	600	750	8.49	16.30	6.87	9.38	5.60	5.53	4.77	3.60	3.06	1.12	2.12	.43
16	640	800	9.06	18.60	7.33	10.70	6.06	6.48	5.09	4.10	3.26	1.27	2.26	.49
17	680	850	9.62	21.00	7.79	12.00	6.44	7.31	5.41	4.63	3.46	1.43	2.40	.55
18	720	900	10.20	23.60	8.25	13.50	6.82	8.20	5.73	5.19	3.67	1.61	2.55	.62
20	800	1,000	11.30	28.90	9.17	16.70	7.58	10.10	6.37	6.42	4.07	1.98	2.83	.76
22	880	1,100	12.40	34.80	10.10	20.30	8.33	12.20	7.00	7.75	4.48	2.40	3.11	.92
24	960	1,200	13.60	41.90	11.00	24.00	9.09	14.60	7.64	9.23	4.89	2.86	3.40	1.10
26	1,040	1,300	14.70	49.00	11.90	28.10	9.85	17.10	8.28	10.80	5.30	3.36	3.68	1.29
28	1,120	1,400	15.80	56.60	12.80	32.60	10.60	19.80	8.91	12.60	5.70	3.89	3.96	1.49
30	1,200	1,500	17.00	65.50	13.70	37.30	11.40	22.90	9.55	14.40	6.11	4.47	4.24	1.71
32	1,280	1,600	18.10	74.20	14.70	42.90	12.10	25.80	10.20	16.50	6.52	5.09	4.53	1.97
34	1,360	1,700	19.20	83.50	15.60	48.30	12.90	29.40	10.80	18.40	6.93	5.74	4.81	2.20
36	1,440	1,800	20.40	94.30	16.50	54.10	13.60	33.10	11.40	20.60	7.33	6.43	5.09	2.47
38	1,520	1,900	21.50	104.60	17.40	60.20	14.40	36.60	12.10	23.10	7.74	7.16	5.38	2.76

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[1] Columns headed A(40) and B(50) respectively refer to the number of miner's inches in the equivalent number of second-feet. For instance, 72 miner's inches are the equivalent of 1.8 second-feet in communities where 40 inches equals 1 second-foot.

TABLE 8.—FOR ESTIMATING THE CAPACITY OF WELL-MADE CONCRETE PIPE LINES OR PRESSURE TUNNELS.

Based on formula $V=C_sH^{0.5}d^{0.625}$ using 0.345 for the coefficient C_s

For instance, 100 second-feet of water, the equivalent of 64.6 million United States gallons per day, will be conveyed in a 48-inch pipe at a velocity of 7.96 feet per second with a loss of head (grade) of 4.22 feet per thousand feet of pipe.

Quantity (Q).	Inside diameter in inches and corresponding area, A, in square feet.														Quantity, millions per day.
	38 A=7.876		**40** A=8.727		**42** A=9.621		**44** A=10.56		**46** A=11.54		**48** A=12.57		**54** A=15.90		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
Sec.-ft.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Gallons.
20	2.54	0.57	2.29	0.44	2.08	0.34	1.89	0.26	1.73	0.21	1.59	0.17	1.26	0.09	12.926
25	3.17	.89	2.86	.68	2.60	.53	2.37	.42	2.17	.33	1.99	.26	1.57	.14	16.157
30	3.81	1.29	3.44	.99	3.12	.76	2.85	.60	2.60	.47	2.39	.38	1.89	.21	19.389
35	4.44	1.76	4.01	1.36	3.64	1.04	3.31	.81	3.03	.64	2.78	.52	2.20	.28	22.621
40	5.08	2.30	4.58	1.75	4.16	1.36	3.79	1.06	3.47	.84	3.18	.67	2.52	.35	25.853
45	5.71	2.90	5.16	2.22	4.68	1.72	4.25	1.35	3.90	1.07	3.58	.85	2.83	.46	29.084
50	6.35	3.59	5.57	2.59	5.20	2.13	4.74	1.67	4.33	1.32	3.98	1.05	3.14	.57	32.316
55	6.98	4.34	6.30	3.32	5.72	2.57	5.21	2.01	4.77	1.60	4.38	1.28	3.46	.59	35.547
60	7.62	5.17	6.88	3.95	6.24	3.06	5.68	2.39	5.20	1.90	4.77	1.51	3.77	.82	38.779
65	8.25	6.06	7.45	4.64	6.76	3.59	6.16	2.82	5.63	2.23	5.17	1.78	4.09	.95	42.010
70	8.89	7.04	8.02	5.37	7.28	4.17	6.63	3.26	6.07	2.59	5.57	2.06	4.40	1.11	45.242
75	9.52	8.07	8.59	6.16	7.80	4.78	7.10	3.74	6.50	2.96	5.97	2.37	4.72	1.28	48.474
80	10.20	9.27	9.17	7.02	8.32	5.44	7.58	4.26	6.93	3.37	6.36	2.69	5.03	1.45	51.705
85	10.80	10.40	9.74	7.92	8.84	6.14	8.05	4.81	7.37	3.82	6.76	3.04	5.35	1.64	54.937
90	11.40	11.60	10.30	8.86	9.36	6.88	8.52	5.39	7.80	4.27	7.16	3.41	5.66	1.84	58.168
100	12.70	14.40	11.50	11.00	10.40	8.50	9.47	6.66	8.67	5.32	7.96	4.22	6.29	2.27	64.631
110	14.00	17.40	12.60	13.20	11.40	10.20	10.40	8.02	9.53	6.38	8.75	5.09	6.92	2.75	71.096
120	15.20	20.60	13.80	15.90	12.50	12.30	11.40	9.65	10.40	7.59	9.55	6.07	7.55	3.27	77.558
130	16.50	24.20	14.90	18.50	13.50	14.30	12.30	11.20	11.30	8.96	10.30	7.12	8.18	3.84	84.021
140	17.80	28.20	16.00	21.40	14.60	16.80	13.30	13.10	12.10	10.30	11.10	8.19	8.80	4.44	90.484
150	19.00	32.10	17.20	24.70	15.60	19.10	14.20	15.00	13.00	11.90	11.90	9.42	9.43	5.10	96.947
160	20.30	36.70	18.30	28.00	16.60	21.60	15.20	17.10	13.90	13.60	12.70	10.70	10.10	5.85	103.410
170	21.60	41.50	19.50	31.80	17.70	24.60	16.10	19.20	14.70	15.20	13.50	12.10	10.70	6.57	109.870
180	22.80	46.30	20.60	35.40	18.70	27.50	17.00	21.40	15.60	17.10	14.30	13.30	11.30	7.33	116.340
190	24.10	51.70	21.80	39.70	19.80	30.80	18.00	24.00	16.50	19.10	15.10	15.20	12.00	8.26	122.800
	60 A=19.64		**66** A=23.76		**72** A=28.27		**78** A=33.18		**84** A=38.48		**90** A=44.18		**96** A=50.26		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
50	2.55	0.33	2.10	0.20	1.77	0.13	1.51	0.08	1.30	0.06	1.13	0.04	0.99	0.03	32.316
55	2.80	.39	2.31	.24	1.94	.15	1.66	.10	1.43	.07	1.24	.05	1.09	.03	35.547
60	3.06	.47	2.53	.28	2.12	.18	1.81	.12	1.56	.08	1.36	.06	1.19	.04	38.779
65	3.31	.55	2.74	.33	2.30	.21	1.96	.14	1.69	.09	1.47	.07	1.29	.05	42.070
70	3.56	.64	2.95	.39	2.48	.25	2.11	.16	1.82	.10	1.58	.08	1.39	.05	45.242
75	3.82	.73	3.16	.45	2.65	.28	2.26	.18	1.95	.12	1.70	.09	1.49	.06	48.474
80	4.07	.83	3.37	.51	2.83	.32	2.41	.21	2.08	.14	1.81	.10	1.59	.07	51.705
85	4.33	.94	3.58	.57	3.01	.36	2.56	.24	2.21	.16	1.92	.11	1.69	.08	54.937
90	4.58	1.06	3.79	.64	3.18	.40	2.71	.27	2.34	.18	2.04	.13	1.79	.09	58.168
95	4.84	1.19	4.00	.71	3.36	.45	2.8[illegible]	.30	2.47	.20	2.15	.14	1.89	.10	61.400
100	5.09	1.30	4.21	.79	3.54	.50	3.01	.33	2.60	.22	2.26	.15	1.99	.11	64.631
110	5.60	1.58	4.63	.96	3.89	.61	3.32	.40	2.86	.27	2.49	.19	2.19	.13	71.095
120	6.11	1.88	5.05	1.14	4.25	.72	3.62	.47	3.12	.32	2.72	.22	2.39	.16	77.558
130	6.62	2.20	5.47	1.34	4.60	.85	3.92	.56	3.38	.38	2.94	.26	2.59	.19	84.021
140	7.13	2.56	5.89	1.55	4.95	.98	4.22	.64	3.64	.44	3.17	.30	2.79	.22	90.484
150	7.64	2.94	6.31	1.78	5.31	1.13	4.52	.74	3.90	.50	3.39	.35	2.98	.25	96.947
160	8.15	3.34	6.73	2.02	5.66	1.28	4.82	.84	4.16	.57	3.62	.40	3.18	.28	103.410
170	8.66	3.78	7.16	2.29	6.01	1.45	5.12	.95	4.42	.64	3.85	.45	3.38	.32	109.870
180	9.17	4.23	7.58	2.56	6.37	1.62	5.43	1.07	4.68	.72	4.07	.50	3.58	.36	116.340
190	9.67	4.71	8.00	2.86	6.72	1.81	5.73	1.19	4.94	.81	4.30	.56	3.78	.40	122.800
200	10.20	5.23	8.42	3.16	7.08	2.01	6.03	1.32	5.20	.89	4.53	.62	3.98	.44	129.260
220	11.20	6.32	9.26	3.83	7.78	2.42	6.63	1.59	5.72	1.08	4.98	.75	4.38	.54	142.190
240	12.20	7.49	10.10	4.55	8.49	2.89	7.23	1.89	6.24	1.29	5.43	.89	4.78	.64	155.120
260	13.20	8.77	10.90	5.30	9.20	3.39	7.84	2.23	6.76	1.51	5.88	1.05	5.17	.75	168.040
280	14.20	10.20	11.80	6.22	9.90	3.93	8.44	2.58	7.28	1.75	6.34	1.22	5.57	.84	180.970
300	15.30	11.80	12.60	7.09	10.60	4.50	9.04	2.96	7.80	2.01	6.79	1.40	5.97	.95	193.890
320	16.30	13.40	13.50	8.13	11.30	5.12	9.64	3.36	8.32	2.29	7.24	1.59	6.37	1.13	206.820
340	17.30	15.10	14.30	9.13	12.00	5.77	10.30	3.84	8.84	2.58	7.70	1.80	6.77	1.28	219.750
360	18.30	16.90	15.20	10.30	12.70	6.46	10.90	4.30	9.36	2.89	8.15	2.01	7.16	1.43	232.670
380	19.30	18.80	16.00	11.40	13.40	7.19	11.50	4.79	9.88	3.22	8.60	2.24	7.56	1.60	245.600

TABLE 8.—FOR ESTIMATING THE CAPACITY OF WELL-MADE CONCRETE PIPE LINES OR PRESSURE TUNNELS—Continued.

For instance, 400 second-feet of water, the equivalent of 259 million U. S. gallons per day, will be conveyed in a 144-inch (12-foot) pipe at a velocity of 3.54 feet per second with a loss of head (grade) of 0.21 feet per thousand feet of pipe.

Quantity (Q).	Inside diameter in inches and corresponding area, A, in square feet.														Quantity, millions per day.
	102 $A=56.74$		**108** $A=63.62$		**114** $A=70.88$		**120** $A=78.54$		**126** $A=86.59$		**132** $A=95.03$		**138** $A=103.9$		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
Sec. ft.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Gals.*
160	2.82	0.21	2.51	0.15	2.26	0.11	2.04	0.09	1.85	0.07	1.68	0.05	1.54	0.04	103
170	3.00	.23	2.67	.17	2.40	.13	2.17	.10	1.96	.08	1.79	.06	1.64	.05	110
180	3.17	.26	2.83	.19	2.54	.14	2.29	.11	2.08	.09	1.89	.07	1.73	.05	116
190	3.35	.29	2.99	.22	2.68	.16	2.42	.12	2.19	.10	2.00	.08	1.83	.06	123
200	3.53	.32	3.14	.24	2.82	.18	2.55	.14	2.31	.11	2.10	.08	1.92	.07	129
220	3.88	.39	3.46	.29	3.10	.22	2.80	.17	2.54	.13	2.32	.10	2.12	.08	142
240	4.23	.46	3.77	.34	3.39	.26	3.06	.20	2.77	.15	2.52	.12	2.31	.09	155
260	4.58	.54	4.09	.40	3.67	.30	3.31	.23	3.00	.18	2.74	.14	2.50	.11	168
280	4.94	.63	4.40	.47	3.95	.35	3.56	.27	3.23	.21	2.95	.16	2.70	.13	181
300	5.29	.72	4.72	.54	4.23	.40	3.82	.31	3.46	.24	3.16	.19	2.89	.15	194
320	5.64	.82	5.03	.61	4.52	.46	4.07	.35	3.69	.27	3.37	.21	3.08	.17	207
340	5.99	.93	5.34	.69	4.80	.52	4.33	.40	3.93	.31	3.58	.24	3.27	.19	220
360	6.35	1.04	5.66	.77	5.08	.58	4.58	.45	4.16	.34	3.79	.27	3.46	.21	233
380	6.70	1.16	5.97	.86	5.36	.65	4.84	.50	4.39	.38	4.00	.30	3.66	.24	246
400	7.05	1.29	6.29	.95	5.64	.72	5.09	.55	4.62	.42	4.21	.33	3.85	.26	253
420	7.40	1.42	6.60	1.05	5.92	.79	5.35	.60	4.85	.47	4.42	.37	4.04	.29	271
440	7.75	1.56	6.92	1.16	6.21	.87	5.60	.66	5.08	.51	4.63	.40	4.24	.32	284
460	8.11	1.70	7.23	1.26	6.49	.95	5.86	.73	5.31	.56	4.84	.44	4.43	.35	297
480	8.46	1.86	7.54	1.37	6.77	1.03	6.11	.79	5.54	.61	5.05	.48	4.62	.38	310
500	8.81	2.01	7.86	1.49	7.05	1.12	6.37	.86	5.77	.66	5.26	.52	4.81	.41	323
550	9.69	2.43	8.64	1.80	7.76	1.36	7.00	1.04	6.35	.80	5.79	.63	5.29	.50	355
600	10.60	2.91	9.43	2.15	8.46	1.61	7.64	1.23	6.93	.96	6.31	.75	5.78	.59	388
650	11.50	3.36	10.20	2.51	9.16	1.90	8.28	1.45	7.51	1.12	6.84	.88	6.26	.70	420
700	12.30	3.92	11.00	2.92	9.88	2.20	8.91	1.68	8.08	1.30	7.37	1.02	6.74	.81	452
750	13.20	4.51	11.80	3.36	10.60	2.53	9.55	1.93	8.66	1.49	7.89	1.17	7.22	.92	485
800	14.10	5.15	12.60	3.83	11.30	2.88	10.20	2.20	9.24	1.70	8.42	1.33	7.70	1.05	517
850	15.00	5.83	13.40	4.32	12.00	3.24	10.80	2.47	9.82	1.92	8.94	1.50	8.18	1.19	549
900	15.90	6.55	14.20	4.87	12.70	3.63	11.50	2.80	10.40	2.15	9.47	1.68	8.66	1.33	582
950	16.80	7.23	14.90	5.35	13.40	4.04	12.10	3.10	11.00	2.41	10.00	1.88	9.14	1.48	614
1,000	17.60	8.02	15.70	5.95	14.10	4.48	12.70	3.41	11.50	3.63	10.50	2.07	9.69	1.64	646
	144 $A=113.1$		**156** $A=132.7$		**168** $A=153.94$		**180** $A=176.71$		**192** $A=201.1$		**204** $A=227.0$		**216** $A=254.5$		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
200	1.77	0.05	1.51	0.02	1.30	0.02	1.13	0.02	0.99	0.01	0.88	0.01	0.79	0.01	129
250	2.21	.08	1.88	.05	1.62	.04	1.41	.03	1.24	.02	1.10	.01	.98	.01	162
300	2.65	.12	2.26	.08	1.95	.05	1.70	.04	1.49	.03	1.32	.02	1.18	.01	194
350	3.09	.16	2.64	.11	2.27	.07	1.98	.05	1.74	.04	1.54	.03	1.38	.02	226
400	3.54	.21	3.01	.14	2.60	.09	2.26	.06	1.99	.05	1.76	.03	1.57	.02	259
450	3.98	.27	3.39	.17	2.92	.12	2.55	.08	2.24	.06	1.98	.04	1.77	.03	291
500	4.42	.33	3.77	.21	3.25	.15	2.83	.10	2.49	.07	2.20	.05	1.97	.04	323
550	4.85	.40	4.15	.26	3.57	.18	3.11	.12	2.74	.09	2.42	.07	2.16	.05	355
600	5.31	.47	4.52	.31	3.90	.21	3.40	.15	2.98	.10	2.64	.08	2.36	.06	388
650	5.75	.56	4.90	.37	4.22	.25	3.68	.17	3.23	.12	2.86	.09	2.55	.07	420
700	6.19	.64	5.28	.42	4.55	.29	3.96	.20	3.48	.14	3.08	.10	2.75	.08	452
750	6.63	.74	5.65	.48	4.87	.33	4.24	.23	3.73	.16	3.30	.12	2.95	.09	485
800	7.07	.84	6.03	.55	5.20	.38	4.53	.26	3.98	.19	3.52	.14	3.14	.10	517
850	7.52	.95	6.41	.63	5.52	.42	4.81	.29	4.23	.21	3.74	.15	3.34	.11	549
900	7.96	1.07	6.78	.70	5.85	.48	5.09	.33	4.48	.24	3.96	.17	3.54	.12	582
950	8.40	1.19	7.16	.78	6.17	.53	5.38	.37	4.72	.26	4.18	.19	3.73	.14	614
1,000	8.84	1.32	7.54	.87	6.50	.59	5.66	.41	4.97	.29	4.41	.21	3.93	.16	646
1,050	9.28	1.45	7.91	.95	6.82	.65	5.94	.45	5.23	.32	4.63	.23	4.12	.17	678
1,100	9.73	1.59	8.29	1.05	7.15	.71	6.22	.49	5.47	.35	4.85	.26	4.32	.19	711
1,150	10.20	1.75	8.67	1.14	7.47	.78	6.51	.56	5.72	.38	5.07	.28	4.52	.21	743
1,200	10.60	1.89	9.04	1.24	7.80	.85	6.79	.59	5.97	.42	5.29	.30	4.71	.23	776
1,250	11.10	2.07	9.42	1.35	8.12	.92	7.07	.64	6.22	.45	5.51	.33	4.92	.24	807
1,300	11.50	2.23	9.80	1.46	8.45	.99	7.36	.69	6.47	.49	5.73	.36	5.11	.26	840
1,400	12.40	2.59	10.60	1.71	9.09	1.15	7.92	.80	6.96	.57	6.17	.41	5.50	.31	905
1,500	13.30	2.98	11.30	1.94	9.74	1.32	8.49	.92	7.46	.65	6.61	.48	5.90	.35	969
1,600	14.20	3.39	12.10	2.23	10.40	1.50	9.05	1.04	7.96	.74	7.05	.54	6.29	.40	1,403
1,700	15.00	3.79	12.80	2.50	11.00	1.68	9.62	1.18	8.46	.84	7.49	.61	6.68	.45	1,098
1,800	15.90	4.26	13.60	2.82	11.70	1.90	10.20	1.32	8.95	.94	7.93	.68	7.07	.51	1,163
1,900	16.80	4.75	14.30	3.15	12.30	2.12	10.80	1.48	9.48	1.05	8.37	.76	7.47	.56	1,228
2,000	17.70	5.27	15.10	3.48	13.00	2.35	11.30	1.62	9.95	1.16	8.81	.85	7.86	.63	1,922

TABLE 9.—FOR ESTIMATING THE CAPACITY OF GLAZED CONCRETE PIPE LINES.

Based on formula $V = C_s H^{0.5} d^{0.625}$ using 0.370 for the coefficient C_s For instance, 140 second-feet of water, the equivalent of 90.5 million United States gallons per day, will be conveyed by a 54-inch pipe at a velocity of 8.80 feet per second with a loss of head (grade) of 3.86 feet per thousand feet of pipe.

Quantity (Q).	Inside diameter in inches and corresponding area, *A*, in square feet.														Quantity, millions per day.
	16 *A*=1.396		**18** *A*=1.767		**20** *A*=2.182		**22** *A*=2.640		**24** *A*=3.142		**30** *A*=4.909		**36** *A*=7.068		
	V	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	
Sec.-ft.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Gals.*
2.0	1.43	0.48	1.13	0.25	0.92	0.15	0.76	0.09	0.64	0.05	......	......	......	......	1.29
2.5	1.79	.73	1.42	.40	1.15	.23	.95	.14	.80	.09	0.51	0.03	......	......	1.62
3.0	2.15	1.06	1.70	.56	1.37	.32	1.14	.20	.95	.12	.61	.03	......	......	1.94
3.5	2.51	1.44	1.98	.77	1.60	.44	1.33	.27	1.11	.17	.71	.05	0.50	0.02	2.26
4.0	2.86	1.87	2.23	.98	1.83	.57	1.52	.36	1.27	.23	.81	.07	.57	.03	2.58
4.5	3.22	2.35	2.55	1.28	2.06	.73	1.70	.44	1.43	.28	.92	.09	.64	.03	2.91
5.0	3.58	2.92	2.83	1.57	2.29	.90	1.89	.55	1.59	.35	1.02	.10	.71	.04	3.23
5.5	3.94	3.54	3.11	1.90	2.52	1.10	2.08	.66	1.75	.42	1.12	.13	.78	.05	3.55
6.0	4.30	4.22	3.40	2.28	2.75	1.30	2.27	.79	1.91	.50	1.22	.16	.85	.06	3.83
6.5	4.66	4.95	3.68	2.67	2.98	1.53	2.46	.93	2.07	.59	1.32	.18	.92	.07	4.20
7.0	5.01	5.73	3.96	3.09	3.21	1.78	2.65	1.08	2.23	.69	1.42	.21	.99	.08	4.52
7.5	5.37	6.58	4.24	3.54	3.44	2.04	2.84	1.23	2.39	.78	1.53	.24	1.06	.10	4.85
8.0	5.72	7.49	4.53	4.04	3.67	2.33	3.03	1.41	2.55	.90	1.63	.28	1.13	.10	5.17
8.5	6.09	8.46	4.81	4.56	3.90	2.62	3.22	1.59	2.71	1.02	1.73	.31	1.20	.12	5.49
9.0	6.45	9.49	5.10	5.13	4.12	2.93	3.41	1.78	2.86	1.12	1.83	.35	1.27	.13	5.82
9.5	6.80	10.60	5.38	5.70	4.35	3.27	3.60	1.99	3.02	1.25	1.94	.39	1.34	.15	6.14
10	7.16	11.70	5.66	6.31	4.58	3.62	3.79	2.20	3.18	1.39	2.04	.43	1.41	.16	6.46
11	7.88	14.20	6.22	7.62	5.04	4.38	4.17	2.67	3.50	1.69	2.24	.52	1.56	.20	7.11
12	8.60	16.90	6.79	9.04	5.50	5.22	4.55	3.17	3.82	2.01	2.44	.62	1.70	.24	7.76
13	9.31	19.80	7.36	10.70	5.96	6.14	4.92	3.71	4.14	2.36	2.65	.73	1.84	.28	8.40
14	10.00	22.90	7.92	12.30	6.42	7.12	5.30	4.30	4.46	2.74	2.85	.84	1.98	.32	9.00
15	10.70	26.30	8.49	14.20	6.87	8.16	5.60	4.81	4.77	3.13	3.06	.97	2.12	.37	9.69
16	11.50	30.10	9.06	16.20	7.33	9.30	6.06	5.63	5.09	3.56	3.26	1.10	2.26	.43	10.30
17	12.20	34.00	9.62	18.20	7.79	10.40	6.44	6.36	5.41	4.02	3.46	1.24	2.40	.48	11.00
18	12.90	38.00	10.20	20.50	8.25	11.70	6.82	7.13	5.73	4.51	3.67	1.40	2.55	.54	11.60
20	14.30	46.60	11.30	25.10	9.17	14.50	7.58	8.78	6.39	5.58	4.07	1.72	2.83	.66	12.90
22	15.80	56.90	12.40	30.20	10.10	17.60	8.33	10.60	7.00	6.74	4.48	2.09	3.11	.80	14.20
24	17.20	67.50	13.60	36.40	11.00	20.80	9.09	12.80	7.64	8.02	4.89	2.49	3.40	.96	15.50
26	18.60	78.90	14.70	42.60	11.90	24.40	9.85	14.90	8.27	9.39	5.30	2.92	3.68	1.12	16.80
28	20.00	91.30	15.80	49.20	12.80	28.30	10.60	17.20	8.91	11.00	5.70	3.38	3.96	1.30	18.10
30	21.50	105.50	17.00	56.90	13.70	32.40	11.40	20.00	9.55	12.50	6.11	3.89	4.24	1.49	19.40
32	22.90	114.60	18.10	64.10	14.70	37.30	12.10	22.40	10.20	14.30	6.52	4.42	4.53	1.71	20.70
34	24.00	135.70	19.20	72.60	15.60	42.00	12.90	25.60	10.80	16.00	6.93	4.99	4.81	1.91	22.00
36	25.80	151.80	20.40	82.00	16.50	47.00	13.60	28.80	11.40	17.90	7.33	5.59	5.08	2.15	23.30
38	27.20	168.60	21.50	90.90	17.40	52.00	14.40	31.80	12.10	20.10	7.74	6.22	5.38	2.40	24.50
	38 *A*=7.876		**40** *A*=8.727		**42** *A*=9.621		**44** *A*=10.56		**46** *A*=11.54		**48** *A*=12.57		**54** *A*=15.90		
	V	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	*V*	*H*	
20	2.54	0.50	2.29	0.38	2.08	0.30	1.89	0.23	1.73	0.18	1.59	0.15	1.26	0.08	12.9
25	3.17	.77	2.86	.59	2.60	.46	2.37	.37	2.17	.29	1.99	.23	1.57	.12	16.2
30	3.81	1.12	3.44	.86	3.12	.66	2.85	.52	2.60	.41	2.39	.33	1.89	.18	19.3
35	4.44	1.53	4.01	1.18	3.64	.90	3.31	.70	3.03	.56	2.78	.45	2.20	.24	22.6
40	5.08	2.00	4.58	1.52	4.16	1.18	3.79	.92	3.47	.73	3.18	.58	2.52	.31	25.9
45	5.71	2.52	5.16	1.93	4.68	1.50	4.26	1.17	3.90	.93	3.58	.74	2.83	.40	29.1
50	6.35	3.12	5.57	2.25	5.20	1.85	4.74	1.45	4.33	1.15	3.98	.91	3.14	.50	32.3
55	6.98	3.77	6.30	2.89	5.72	2.23	5.21	1.75	4.77	1.39	4.38	1.11	3.46	.60	35.5
60	7.62	4.49	6.88	3.43	6.24	2.66	5.68	2.08	5.20	1.65	4.77	1.31	3.77	.71	38.8
65	8.25	5.27	7.45	4.03	6.76	3.12	6.16	2.45	5.63	1.94	5.17	1.55	4.09	.83	42.0
70	8.89	6.12	8.02	4.67	7.28	3.62	6.63	2.83	6.07	2.25	5.57	1.79	4.40	.96	45.2
75	9.52	7.02	8.59	5.36	7.80	4.16	7.10	3.25	6.50	2.57	5.97	2.06	4.72	1.11	48.5
80	10.20	8.06	9.17	6.10	8.32	4.73	7.58	3.70	6.93	2.93	6.36	2.34	5.03	1.26	51.7
85	10.80	9.04	9.74	6.88	8.84	5.34	8.05	4.18	7.37	3.32	6.76	2.64	5.35	1.42	54.9
90	11.40	10.10	10.30	7.70	9.36	5.98	8.52	4.69	7.80	3.71	7.16	2.96	5.66	1.60	58.2
100	12.70	12.50	11.50	9.56	10.40	7.39	9.47	5.79	8.67	4.62	7.96	3.67	6.29	1.97	64.6
110	14.00	15.10	12.60	11.50	11.40	8.87	10.40	6.97	9.53	5.55	8.75	4.42	6.92	2.39	71.1
120	15.20	17.90	13.80	13.80	12.50	10.70	11.40	8.89	10.40	6.60	9.55	5.22	7.55	2.84	77.6
130	16.50	21.00	14.90	16.10	13.50	12.40	12.30	9.74	11.30	7.79	10.30	6.19	8.18	3.34	84.0
140	17.80	24.50	16.00	18.60	14.60	14.60	13.30	11.40	12.10	8.96	11.10	7.12	8.80	3.86	90.5
150	19.00	27.90	17.20	21.50	15.60	16.60	14.20	13.00	13.00	10.30	11.90	8.19	9.43	4.43	96.9
160	20.30	31.90	18.30	24.30	16.60	18.80	15.20	14.80	13.90	11.80	12.70	9.30	10.10	5.09	103.0
170	21.60	36.10	19.50	27.60	17.70	21.40	16.10	16.70	14.70	13.20	13.50	10.50	10.70	5.71	110.0
180	22.80	40.20	20.60	30.80	18.70	23.90	17.00	18.60	15.60	14.90	14.30	11.80	11.30	6.37	116.0
190	24.10	44.90	21.80	34.50	19.80	26.80	18.00	20.90	16.50	16.60	15.10	13.20	12.00	7.18	123.0

TABLE 9.—FOR ESTIMATING THE CAPACITY OF GLAZED CONCRETE PIPE LINES—Continued.

For instance, 400 second-feet of water, the equivalent of 259 million United States gallons per day, will be conveyed by a 120-inch pipe at a velocity of 5.09 feet per second with a loss of head (grade) of 0.48 foot per thousand feet of pipe.

Quantity (Q).	Inside diameter in inches and corresponding area, A, in square feet.														Quantity, millions per day.
	60 $A=19.64$		**66** $A=23.76$		**72** $A=28.27$		**78** $A=33.18$		**84** $A=38.48$		**90** $A=44.18$		**96** $A=50.26$		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
Sec.-ft.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Gals.*
50	2.55	0.29	2.10	0.17	1.77	0.11	1.51	0.07	1.30	0.05	1.13	0.03	0.99	0.03	32.3
55	2.80	.34	2.31	.21	1.94	.13	1.66	.09	1.43	.06	1.24	.04	1.09	.03	35.5
60	3.06	.41	2.53	.24	2.12	.16	1.81	.10	1.56	.07	1.36	.05	1.19	.03	38.8
65	3.31	.48	2.74	.29	2.30	.18	1.96	.12	1.69	.08	1.47	.06	1.29	.04	42.0
70	3.56	.56	2.95	.34	2.48	.22	2.11	.14	1.82	.09	1.58	.07	1.39	.04	45.2
75	3.82	.63	3.16	.39	2.65	.24	2.26	.16	1.95	.10	1.70	.08	1.49	.05	48.5
80	4.07	.72	3.37	.44	2.83	.28	2.41	.18	2.08	.12	1.81	.09	1.59	.06	51.7
85	4.33	.82	3.58	.50	3.01	.31	2.56	.21	2.21	.14	1.92	.10	1.69	.07	54.9
90	4.58	.92	3.79	.56	3.18	35	2.71	.23	2.34	.16	2.04	.11	1.79	.08	58.2
95	4.84	1.03	4.00	.62	3.36	.39	2.86	.26	2.47	.17	2.15	.12	1.89	.09	61.4
100	5.09	1.13	4.21	.69	3.54	.43	3.01	.29	2.60	.19	2.26	.13	1.99	.10	64.6
110	5.60	1.37	4.63	.83	3.89	.53	3.32	.35	2.86	.23	2.49	.16	2.19	.12	71.1
120	6.11	1.63	5.05	.99	4.25	.62	3.62	.41	3.12	.28	2.72	.19	2.39	.14	77.6
130	6.62	1.91	5.47	1.16	4.60	.74	3.92	.49	3.38	.33	2.94	.23	2.59	.17	84.0
140	7.13	2.22	5.89	1.35	4.95	.85	4.22	.56	3.64	.38	3.17	.26	2.79	.19	90.6
150	7.64	2.56	6.31	1.55	5.31	.98	4.52	.64	3.90	.43	3.39	.30	2.98	.22	96.9
160	8.15	2.90	6.73	1.76	5.66	1.11	4.82	.73	4.16	.50	3.62	.35	3.18	.24	103.0
170	8.66	3.29	7.16	1.99	6.01	1.26	5.12	.82	4.42	.56	3.85	.39	3.38	.28	110.0
180	9.17	3.68	7.58	2.22	6.37	1.41	5.43	.93	4.68	.62	4.07	.43	3.58	.31	116.0
190	9.67	4.10	8.00	2.49	6.72	1.57	5.73	1.03	4.94	.70	4.30	.49	3.78	.35	123.0
200	10.20	4.55	8.42	2.75	7.08	1.75	6.03	1.15	5.20	.77	4.53	.54	3.98	.38	129.0
220	11.20	5.49	9.26	3.33	7.78	2.10	6.63	1.38	5.72	.94	4.98	.65	4.38	.47	142.0
240	12.20	6.51	10.10	3.96	8.49	2.51	7.23	1.64	6.24	1.12	5.43	.77	4.78	.56	155.0
260	13.20	7.62	10.90	4.61	9.20	2.95	7.84	1.94	6.76	1.31	5.88	.91	5.17	.65	168.0
280	14.20	8.87	11.80	5.41	9.90	3.42	8.44	2.24	7.28	1.52	6.34	1.06	5.57	.73	181.0
300	15.30	10.20	12.60	6.23	10.60	3.91	9.04	2.57	7.80	1.75	6.79	1.22	5.97	.83	194.0
320	16.30	11.60	13.50	7.07	11.30	4.45	9.64	2.92	8.32	1.99	7.24	1.38	6.37	.98	207.0
340	17.30	13.10	14.30	7.94	12.00	5.02	10.30	3.34	8.84	2.24	7.70	1.56	6.77	1.11	220.0
360	18.30	14.70	15.20	8.96	12.70	5.62	10.90	3.74	9.36	2.51	8.15	1.75	7.16	1.24	233.0
380	19.30	16.30	16.00	9.91	13.40	6.25	11.50	4.16	9.88	2.80	8.60	1.95	7.56	1.39	246.0
	102 $A=56.74$		**108** $A=63.62$		**114** $A=70.88$		**120** $A=78.54$		**126** $A=86.59$		**132** $A=95.03$		**138** $A=103.9$		
	V	H	V	H	V	H	V	H	V	H	V	H	V	H	
160	2.82	0.18	2.51	0.13	2.26	0.10	2.04	0.08	1.85	0.06	1.68	0.04	1.54	0.03	103
170	3.00	.20	2.67	.15	2.40	.11	2.17	.09	1.96	.07	1.79	.05	1.64	.04	110
180	3.17	.23	2.83	.17	2.54	.12	2.29	.10	2.08	.08	1.89	.06	1.73	.04	116
190	3.35	.25	2.99	.19	2.68	.14	2.42	.11	2.19	.09	2.00	.07	1.83	.05	123
200	3.53	.28	3.14	.21	2.82	.16	2.55	.12	2.31	.10	2.10	.07	1.92	.06	129
220	3.88	.34	3.46	.25	3.10	.19	2.80	.15	2.54	.11	2.32	.09	2.12	.07	142
240	4.23	.40	3.77	.30	3.39	.23	3.06	.17	2.77	.13	2.52	.10	2.31	.08	155
260	4.58	.47	4.09	.35	3.67	.26	3.31	.20	3.00	.16	2.74	.12	2.50	.10	168
280	4.94	.55	4.40	.41	3.95	.30	3.56	.23	3.23	.18	2.95	.14	2.70	.11	181
300	5.29	.62	4.72	.47	4.23	.35	3.82	.27	3.46	.21	3.16	.16	2.89	.13	194
320	5.64	.71	5.03	.53	4.52	.40	4.07	.30	3.69	.23	3.37	.18	3.08	.15	206
340	5.99	.81	5.34	.60	4.80	.45	4.33	.35	3.93	.27	3.58	.21	3.27	.16	220
360	6.35	.90	5.66	.67	5.08	.50	4.58	.39	4.16	.30	3.79	.23	3.46	.18	233
380	6.70	1.01	5.97	.75	5.36	.56	4.84	.43	4.39	.33	4.00	.26	3.66	.21	246
400	7.05	1.12	6.29	.82	5.64	.62	5.09	.48	4.62	.36	4.21	.29	3.85	.23	259
420	7.40	1.23	6.60	.91	5.92	.69	5.35	.52	4.85	.41	4.42	.32	4.04	.25	271
440	7.75	1.36	6.92	1.01	6.20	.76	5.60	.57	5.08	.44	4.63	.35	4.24	.28	284
460	8.11	1.48	7.23	1.10	6.49	.82	5.86	.63	5.31	.49	4.84	.38	4.43	.30	297
480	8.46	1.62	7.54	1.19	6.72	.90	6.11	.69	5.54	.53	5.05	.42	4.62	.33	310
500	8.81	1.75	7.86	1.30	7.05	.97	6.37	.75	5.77	.57	5.26	.45	4.81	.36	323
550	9.69	2.11	8.64	1.56	7.76	1.18	7.00	.90	6.35	.70	5.79	.55	5.29	.43	355
600	10.60	2.53	9.43	1.87	8.46	1.40	7.64	1.07	6.93	.83	6.31	.65	5.78	.51	388
650	11.50	2.92	10.20	2.18	9.16	1.65	8.28	1.26	7.51	.97	6.84	.76	6.26	.60	420
700	12.30	3.41	11.00	2.54	9.88	1.91	8.91	1.46	8.08	1.13	7.37	.89	6.74	.70	452
750	13.20	3.92	11.80	2.92	10.60	2.20	9.55	1.68	8.66	1.30	7.89	1.02	7.22	.80	485
800	14.10	4.48	12.60	3.30	11.30	2.50	10.20	1.91	9.24	1.48	8.42	1.16	7.71	.91	517
850	15.00	5.07	13.40	3.76	12.00	2.82	10.80	2.15	9.82	1.67	8.94	1.30	8.18	1.03	549
900	15.90	5.69	14.20	4.23	12.70	3.16	11.50	2.43	10.40	1.87	9.47	1.46	8.66	1.16	582
950	16.80	6.28	14.90	4.66	13.40	3.51	12.10	2.70	11.00	2.10	10.00	1.63	9.14	1.29	614
1,000	17.60	6.97	15.70	5.17	14.10	3.90	12.70	2.96	11.50	3.16	10.50	1.80	9.63	1.43	646

TABLE 10.—VELOCITIES, IN FEET PER SECOND, AS COMPUTED BY VARIOUS FORMULAS FOR GIVEN SIZES OF PIPE WITH GIVEN FRICTION HEADS.

1	2	Velocities, in feet per second, by formulas.																	
Diameter (d).	Friction head (H).	Scobey. Coefficient (C_s).				Williams-Hazen. Coefficient (C_w).						Kutter. Coefficient (n).					Moritz. Coefficient (C_m).		20
		3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	Scobey.[1]
		0.267	0.310	0.345	0.370	90	100	110	120	130	140	0.014	0.013	0.012	0.0115	0.011	1.24	1.31	
Ins.	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*
6	2.0	1.16	1.35	.....	.....	1.11	1.24	1.36	1.48	1.61	....	1.01	1.09	1.24	......	.....	1.45	1.53	1.52
6	5.0	1.83	2.10	.....	.....	1.84	2.04	2.25	2.45	2.65	....	1.61	1.73	1.98	......	.....	2.37	2.50	2.52
6	9.0	2.46	2.85	.....	.....	2.52	2.80	3.08	3.36	3.64	....	2.15	2.33	2.68	......	.....	3.28	3.47	3.50
12	1.0	1.26	1.47	1.63	1.75	1.19	1.32	1.45	1.58	1.72	1.85	1.22	1.35	1.48	1.56	1.65	1.58	1.67	1.62
12	5.0	2.82	3.28	3.65	3.91	2.84	3.15	3.47	3.78	4.10	4.40	2.74	3.02	3.36	3.54	3.75	3.86	4.08	3.96
12	9.0	3.79	4.39	4.89	5.25	3.89	4.33	4.76	5.19	5.62	6.06	3.70	4.05	4.50	4.72	5.00	5.36	5.66	5.48
36	0.2	1.12	1.30	1.45	1.55	0.99	1.11	1.22	1.33	1.44	1.55	1.19	1.34	1.48	1.53	1.66	1.39	1.47	1.35
36	1.0	2.51	2.91	3.24	3.47	2.38	2.64	2.91	3.16	3.43	3.69	2.76	3.01	3.31	3.49	3.73	3.40	3.59	3.31
36	5.0	5.61	6.51	7.24	7.77	5.66	6.30	6.92	7.55	8.18	8.81	6.20	6.74	7.48	7.85	8.30	8.33	8.80	8.08
72	0.2	.....	.....	2.21	2.40	1.54	1.71	1.88	2.06	2.23	2.40	1.96	2.12	2.31	2.43	2.59	2.26	2.39	2.12
72	1.0	.....	.....	5.00	5.36	3.68	4.08	4.49	4.89	5.31	5.71	4.46	4.83	5.27	5.51	5.79	5.52	5.84	5.19
72	2.0	.....	.....	7.07	7.58	5.35	5.94	6.54	7.14	7.73	8.32	6.31	6.86	7.47	7.80	8.20	8.14	8.59	7.63
96	0.2	.....	.....	2.67	2.87	1.85	2.06	2.26	2.46	2.67	2.88	2.39	2.58	2.81	2.94	3.07	2.77	2.93	2.56
96	0.6	.....	.....	4.62	4.91	3.34	3.72	4.08	4.46	4.83	5.21	4.18	4.51	4.90	5.11	5.37	5.20	5.39	4.70
96	1.0	.....	.....	5.98	6.41	4.41	4.90	5.38	5.78	6.37	6.86	5.40	5.84	6.34	6.61	6.95	6.77	7.15	6.26
120	0.2	.....	.....	3.08	3.30	2.13	2.37	2.60	2.84	3.08	3.31	2.77	3.00	3.24	3.38	3.57	3.24	3.42	2.96
120	0.6	.....	.....	5.37	5.71	3.85	4.28	4.71	5.14	5.57	5.98	5.82	5.20	5.65	5.89	6.18	5.96	6.30	5.45
120	1.0	.....	.....	6.88	7.37	5.08	5.64	6.20	6.76	7.32	7.90	6.23	6.73	7.30	7.62	7.98	7.92	8.36	7.24
144	0.2	.....	.....	3.45	3.70	2.39	2.65	2.92	3.18	3.45	3.71	3.13	3.37	3.65	3.80	3.99	3.65	3.85	3.33
144	0.6	.....	.....	5.97	6.40	4.32	4.80	5.27	5.75	6.23	6.70	5.43	5.84	6.34	6.61	6.92	6.76	7.13	6.14
144	1.0	.....	.....	7.71	8.26	5.69	6.32	6.96	7.58	8.21	8.85	7.00	7.55	8.18	8.53	8.93	8.95	9.45	8.15

[1] Based on formula for velocity in average wood-stave pipe; $V=1.62\,D^{.65}\,H^{.555}$. (See Dept. Bul. 376, p. 7.)

COMPARISON OF THE VARIOUS FORMULAS.

For the reason that there are at least four distinct classes of concrete pipes, considered from a capacity standpoint, it was not feasible to make a percentage or graphical comparison between the recommended formula, with varying coefficients and the other formulas mentioned on pages 5 to 8. Of these other formulas, some consider the influences of varying surfaces by means of coefficients, while others are inelastic and were offered for "clean pipes," regardless of materials.

The point that must be kept continually in mind is that concrete pipes offer a greater range of interior surfaces, due to their initial construction, than pipes of any other material, considered by itself, or pipes of all other materials, considered together. These concrete surfaces are almost unbelievably different, aside from all fouling by growths, slimes, or erosion.

In the following discussion classes 1, 2, 3, and 4 will be considered as was described on page 8, in connection with the new formula, the latter being considered as the base from which a comparison is made.

The values of the various coefficients are for favorable conditions, and factors of safety, as listed on page 54 should be used in design.

For a comparison of velocities, as computed from various formulas, for various sizes of pipe and for varying friction heads, the reader is referred to table 10, page 62.

Again taking up the various formulas mentioned on pages 5 to 7; the Chezy formula (4, p. 6), will not be considered directly, but in its modified form, known as Kutter's formula (5, p. 6).

THE KUTTER FORMULA.

In discussing the Moritz experiments with reference to the value of n in Kutter's formula, Hering states[1] that he "recognized as well as did Mr. Kutter himself, almost at the outset, that n was not to be considered a precise and unvarying constant, although it was more nearly so than any other constant before proposed."

The fact that n does vary has been understood by hydraulicians specializing in work involving the Kutter formula; but notwithstanding this the tables and diagrams which have been accepted as standard have assigned values of n to certain degrees of roughness without reference to other conditions. The usual understanding regarding variation occurring in the value for n has been that n is less in large channels than in small ones.

In the case of pipes an opposite effect is noted; that is, the value of n becomes greater as the value of R (which is directly proportional to the diameter) becomes greater. The variations in the proper value of n to assume in the design of pipe become so complicated that the Kutter formula had better be abandoned in favor of the exponential type of formula. This would leave the Kutter formula for its original intended purpose, that of design of open channels, for which it is eminently fitted.[2]

However, for those who wish to use the Kutter formula, the following suggestions are made. Note that the value of n changes with diameter. If velocities are to exceed 8 feet per second, the next lower value of n may be safely used. The values of n given presuppose the use of some factor of safety about as given on page 54:

Class 1. $n=0.013$ for pipes up to 10 inches in diameter.
$n=0.014$ for pipes from 12 to 24 inches in diameter.
$n=0.015$ for pipes from 26 to 42 inches in diameter.
Class 2. $n=0.013$ for pipes up to 36 inches in diameter.
Class 3. $n=0.012$ for pipes up to 24 inches in diameter.
$n=0.013$ for pipes more than 24 inches in diameter.
Class 4. $n=0.011$ for pipes from 12 to 24 inches in diameter.
$n=0.0115$ for pipes from 26 to 48 inches in diameter.
$n=0.012$ for pipes more than 50 inches in diameter

In order to solve problems involving the Kutter formula, Plate XII is given. As being applicable to open channels rather than closed pipes, the hydraulic radius is given instead of the diameter, but the diagram may be used directly for pipes, since $R=\frac{D}{4}$, that is, the hydraulic radius of a 6-foot pipe is $\frac{6}{4}=1.5$ feet.

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[1] Trans. Amer. Soc. Civ. Engin., 74 (1911), p. 459.

[2] The Flow of Water in Irrigation Channels, by Fred. C. Scobey, U. S. Dept. Agr. Bul. 194, p. 60.

THE WEISBACH FORMULA (6, P. 6).

For those who wish to use this rather popular textbook formula, the following values of f are suggested, presupposing the use of some factor of safety about as listed on page 54.

Class 1. $f=0.040$ for pipes up to 24 inches in diameter.
$f=0.030$ for pipes from 26 to 42 inches in diameter.
Class 2. $f=0.030$ for pipes up to 18 inches in diameter.
$f=0.025$ for pipes from 20 inches to 36 inches in diameter.
Class 3. $f=0.020$ for pipes up to 48 inches in diameter.
$f=0.015$ for pipes more than 48 inches in diameter.
Class 4. $f=0.015$ for pipes up to 48 inches in diameter.
$f=0.012$ for pipes more than 48 inches in diameter.

THE WILLIAMS-HAZEN FORMULA (7, p. 6).

This formula, being made elastic by varying coefficients, applies very closely within the usual range of velocities. The following values of C_w are suggested, presupposing the use of factors of safety as listed on page 54. For velocities exceeding 5 feet per second use a coefficient of C_w about 10 lower.

Class 1. $C_w=90$.
Class 2. $C_w=110$.
Class 3. $C_w=120$.
Class 4. $C_w=140$.

THE MORITZ FORMULA (9, p. 7).

The results of the experiments show that the lines of differentiation between classes of concrete surface may be slightly at variance with those suggested by Moritz (see p. 7). A pipe constructed in place with steel forms may have very appreciable shoulders at the ends of the "set-ups," while a jointed pipe may be very smooth at the joints as well as through each unit.

As shown on Plate VI and on page 47, the exponent of D, the velocity is more nearly 2 than 1.80; therefore the application of the Moritz formula changes with the velocities rather than with the diameters, since his exponent of d or D, the diameter, practically agrees with that of the writer.

Using the classification as listed on page 7 and presupposing the use of factors of safety about as given on page 54, the coefficients in the Moritz formula are about as follows:

Class 1. C_m=0.90 for velocities up to 3 feet per second.
Class 2. C_m=1.15 for velocities up to 3 feet per second.
    1.10 for velocities from 3 to 6 feet per second.
Class 3. C_m=1.25 for velocities less than 2 feet per second.
    1.20 for velocities between 2 and 4 feet per second.
    1.15 for velocities between 4 and 6 feet per second.
    1.10 for velocities between 6 and 8 feet per second.
Class 4. C_m=1.35 for velocities less than 2 feet per second.
    1.30 for velocities between 2 and 4 feet per second.
    1.20 for velocities between 4 and 6 feet per second.

CAPACITY OF CONCRETE PIPE COMPARED WITH THAT OF WOOD-STAVE, CAST-IRON, AND RIVETED STEEL PIPE.

Since there are so many classes of concrete pipe, a table showing percentage comparisons between the capacity of each class of pipe with the capacity of pipes of each of the other materials at various ages would be extensive and confusing, so Table 10 may be used. This table gives velocities; capacity for a given size pipe is proportional to velocity.

In a study of this table, the concrete pipe under consideration will classify under columns 3, 4, 5, or 6, according to the class descriptions given on page 7. Average wood-stave pipe comes under column 20. Cast iron and riveted steel come under columns 8, 9, or 11, according to age of the pipe. The coefficients in their formula recommended by Hazen and Williams for new cast iron, new riveted steel, 10-year-old cast iron, 20-year-old cast iron, and 10-year-old riveted steel are, respectively, 130, 110, 110, 100, and 100.

PART 2. FLOW OF WATER IN GRADE LINE PIPES.

PIPES PARTIALLY FILLED.

All available data bearing upon the capacity of concrete pipes and other covered conduits flowing partially full are summarized in Table 11, page 68.

The coefficient of retardation has been computed for five of the best known formulas in use in this country for the design of open channels. The formulas considered are the Chezy formula (No. 3, p. 6); the Kutter formula (No. 5, p. 6); the Williams-Hazen formula (No. 7, p. 6); the Manning formula,[1]

$$V=\frac{1.486}{n}R^{0.67}S^{0.5} \qquad (20)$$

in which n has approximately the same values as in the Kutter formula; the Bazin (1897) formula,

$$V=\frac{157.6}{1+\frac{m}{\sqrt{R}}}R^{0.5}S^{0.5} \qquad (21)$$

For the smoothest cement channels a value of 0.109 for m is suggested in a table found in many textbooks, probably suggested by the first experiment on the Sudbury Conduit. See page 69. A glance at column 17, Table 11, shows that the value of m changes very rapidly with comparatively small changes of surface and of the various hydraulic elements. The writer believes that the formulas of Kutter, Manning, and Williams-Hazen can be applied with much more assuredness than that of Bazin, if a constant retardation factor is to be used for a given surface.

As the pipes and conduits are not under pressure, but for the most part are laid on an even gradient, where the hydraulic grade line lies parallel to and just under the intrados, it is perfectly proper to regard the pipe as an open channel.

In the opinion of the writer the Kutter formula appears to apply, and as this formula is undoubtedly the one most used in this country recommended values of n will be given.

$n=0.0115$ for glazed pipes and conduits carrying filtered water or water from which deposits or growths do not accrue. The pipes to be practically perfect in both surface and joints.

$n=0.012$ for well-made pipes and conduits with first-class joints, smooth monolithic pipes or tunnels when new and clean. The surface to class as good, but not the equal of glazing. A surface such as is obtained by a "wash coat." To be free from shoulders and other obstructions. To convey water from which deposits are not to be expected. May be more freely assumed where high velocities will be attained.

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[1] Robert Manning first offered his formula in 1889 (Trans. Civil Engineers of Ireland, 1890, Book No. 8, p. 175) in a slightly different form than as now accepted. It has been used extensively by some English and Canadian engineers, but has caused little comment in the United States until within the past two or three years. It is much simpler of solution than the Kutter formula, and there does not appear to be any sacrifice in accuracy if the engineer uses the same values of n to which he has become accustomed. Further comments upon the Manning formula are found under the following citations: The Design of Channels for Irrigation or Drainage, by R. B. Buckley, London, 1911, p. 10; Engin. News, June 17, 1915, p. 1171; Engin. Rec., vol. 75, Mar. 10, 1917, p. 395; Engin. News-Rec., vol. 79, Aug. 9, 1917, p. 277; *id.*, vol. 82, Mar. 13, 1919, p. 536; *id.*, Apr. 3, 1919, p. 685. Prof. H. W. King, of Michigan University, conducted computations that convinced him that the Manning formula is to be preferred to that of Kutter. (Handbook of Hydraulics, by H. W. King, New York, 1918, pp. 198, 403.)

$n=0.013$ for well-made pipes, carefully jointed or monolithic without appreciable shoulders, for waters containing a small amount of sewage. May be used also for designing sewers where conditions are such that high velocities may be attained with flushing streams. Applicable to storm sewers which carry but little deposit-creating material at peak load, but which may have a heavy deposit of grease at the high-water line of ordinary sewage flow. Recommended by Metcalf and Eddy for "concrete sewers under good ordinary conditions of work."[1]

The values of n in the Manning formula are sufficiently close to those in the Kutter formula that the same values may be used by engineers partial to the Manning formula. The chief advantage of the latter is its simplicity of computation, but as the Kutter formula is practically never computed outside of a schoolroom—diagrams being quite generally used—this objection is not material.

For those who prefer the William-Hazen formula it would appear that values for C_w of 140, 130, and 120 will quite closely apply to conditions as described for values of n of 0.0115, 0.012, and 0.013, respectively. Figure 6, offered for the general solution of problems by means of the Williams-Hazen formula, may be used for the design of open channels if the value of the hydraulic radius, R, of the prospective water section in the open pipe, be computed into the terms of the equivalent circular section of a full pipe. The value of D, the diameter of the equivalent full pipe, may be computed since $D=4R$.

EXPLANATORY NOTES ON TABLE 11.

Table 11 is similar to Table 3, but contains the data for experiments made on pipes and conduits while they were but partially filled; that is, the surface of the water was exposed to the air. In other words, the pipes and conduits were "flow lines" rather than "pressure lines." With the following additions the explanation for Table 3, page 20, applies to this table also.

Column 2. FF refers to F. C. Finkle, Los Angeles, Calif.
AFB refers to A. F. Bruce, Glasgow, Scotland.
E refers to Boyd Ehle, engineer on construction of Victoria Aqueduct.
JBL refers to J. B. Lippincott, Los Angeles, Calif.
C refers to the late F. C. Coffin.
M refers to F. F. Moore, designing engineer, New York Board of Water Supply.
H refers to Theodore Horton, Albany, N. Y.
P refers to —— —— Perrone, Italy.
S, as before, refers to the writer.

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[1] American Sewerage Practice, 1st ed., New York, 1914, vol. 1, p. 94.

TABLE 11.—*Elements of experiments on partially filled concrete pipes and conduits for the determination of the retardation factors in various formulas.*

1	2	3	4	5	6	7	8	9	10	11	12	13	Coefficients of retardation.				
													14	15	16	17	18
Pipe No.	Experimenter and year.	Observation.	Inside diameter.	Name and description of pipe.	Length of reach tested.	Maximum depth.	Area of water section.	Wetted perimeter.	Hydraulic mean radius.	Discharge per second.	Mean velocity per second.	Hydraulic grade per foot.	Chezy.	Kutter.	Manning.	Bazin (1897).	Williams-Hazen.
			d		*L*		*A*	*P*	*R*	*Q*	*V*	*s*	*C*	*n*	*n'*	*m*	C_w
			In.		*Feet.*	*Feet.*	*Sq. ft.*	*Feet.*	*Feet.*	*Cu. ft.*	*Feet.*	*Feet.*					
49	S 37, 1915...	5	10	Jointed concrete pipe chute, Tieton project, U. S. Reclamation Service, Washington; falls 87.6 feet in 935 feet.	935	....	0.161	....	0.156	1.72	10.68	0.0937	88.6	0.0116	0.0125	0.307	94
		4					.202	....	.178	2.36	11.67	.0936	90.6	.0119	.0125	.311	95
		3					.269	....	.206	3.45	12.85	.0936	92.8	.0118	.0126	.317	95
		2					.310	....	.221	4.15	13.40	.0937	93.3	.0120	.0127	.324	94
		6					.316	....	.223	4.29	13.59	.0937	94.3	.0119	.0126	.317	95
50*a*	FF 1.......	......	22	Mill Creek Power Plant No. 2...	....	1.75	2.60	5.01	.518	9.80	3.77	.002	117	.0116	.0114	.249	124
51*a*	FF 2.......	......	31	Mill Creek Power Plant No. 3...	....	2.46	5.15	6.99	.736	19.72	3.83	.002	100	.0142	.0142	.497	101
52	S 41, 1917...	5	36	Central Colorado Power Co., trunk line near Nederland, Colo.; jointed concrete pipe, laid on a uniform grade as a flow line; nominal slope 0.005.	1,977	1.16	2.46	3.97	.620	16.10	6.56	.00507	117	.0121	.0117	.273	117
		3				1.73	4.16	5.08	.821	31.70	7.62	.00501	119	.0124	.0121	.296	114
		2				2.17	5.04	5.96	.846	40.90	8.11	.00496	125	.0119	.0115	.237	120
		4				2.33	5.85	6.41	.898	48.20	8.25	.00512	122	.0123	.0120	.279	116
		1				2.37	5.93	6.52	.906	49.50	8.34	.00508	123	.0123	.0119	.274	117
53*a*	FF 3, 1904, 1908.	1	36	Lytle Power Plant, California...	....	2.88	6.97	8.29	.840	20.10	2.88	.001	100	.0145	.0145	.538	102
		2		Same pipe, 4 years later.........	....	2.88	6.97	8.29	.840	21.20	3.04	.001	105	.0138	.0138	.460	107
54*a*	E 1, 1915....	1	42	Victoria Aqueduct, B. C., Canada; tests made on flow line when new; pipe, including joints, exceedingly smooth; water clear; alignment sinuous.	800	.72	1.42	3.27	.434	3.36	2.37	.001	114	.0115	.0114	.255	127
		2				1.21	2.97	4.39	.677	9.30	3.13	.001	121	.0118	.0116	.253	127
		3				1.67	4.52	5.34	.846	16.92	3.74	.001	129	.0118	.0112	.205	132
		4				2.17	6.25	6.34	.986	26.00	4.16	.001	132	.0116	.0112	.188	133
		5				Full.	9.62	10.99	.875	40.59	4.22	1 .001	143	.0106	.0102	.098	145
55*a*	E 2, 1915....	1	42	Victoria Aqueduct, B. C., Canada; same general description as for above; this reach near outlet, 27 miles from that above, which is near inlet.	800	.67	1.28	3.05	.420	2.78	2.17	.001	106	.0121	.0121	.315	119
		2				1.15	2.74	4.25	.644	8.52	3.11	.001	122	.0116	.0113	.229	130
		3				1.62	4.34	5.23	.830	16.32	3.76	.001	130	.0114	.0110	.189	134
		4				2.08	5.96	6.16	.967	25.23	4.23	.001	136	.0113	.0108	.154	137
		5				3.00	8.78	7.68	1.143	39.78	4.53	.001	134	.0117	.0112	.187	132
56	S 49, 1916...	5	42	Victoria Aqueduct, B. C., Canada; longer reach near inlet; tested 15 months after those above.	1,986	1.26	3.11	4.50	.691	9.69	3.11	.00105	115	.0124	.0121	.307	121
		6				1.30	3.24	4.57	.710	11.45	3.53	.00103	132	.0110	.0108	.183	137
		9				1.65	4.47	5.30	.843	18.86	4.22	.00104	142	.0106	.0101	.095	146
		3				1.88	5.27	5.76	.916	24.34	4.62	.00106	148	.0104	.0099	.062	150
		4				2.08	5.94	6.20	.958	27.99	4.71	.00108	146	.0105	.0101	.076	147

58*a*	JBL 1......	1	......	San Gabriel Tunnel No. 15, Calif.	446	3.13	14.08	10.71	1.310	70.60	5.01	.00126	123	.0128	.0126	.320	118
59*a*	JBL 2......	2	......	San Gabriel Tunnel No. 23, Calif.	318	3.50	15.73	11.50	1.370	74.64	4.74	.00082	142	.0113	.0111	.133	136
60*a*	JBL 5......	5	......	Old Los Angeles Aqueduct, Calif.	700	....	5.00	6.12	.817	13.55	2.71	.00051	133	.0111	.0106	.170	140
		6					5.16	6.22	.830	14.51	2.81	.00051	137	.0109	.0106	.140	144
61*a*	FB........	......	......	Loch Katrine Aqueduct, Glasgow; a basket-handle-shaped conduit, 9 feet high by 10 feet maximum width; concrete deposited against soaped wood forms made of tongue and groove lumber; reach straight.	6,760	1.72	14.20	....	1.227	26.62	1.87	.000182	125	.0123	.0122	.289	131
						2.16	18.30	....	1.471	37.86	2.07	.000182	127	.0125	.0125	.299	128
						2.16	18.30	....	1.471	38.57	2.11	.000182	129	.0123	.0123	.270	131
						2.22	18.70	....	1.490	41.42	2.22	.000182	135	.0119	.0117	.295	137
						2.21	18.80	....	1.498	39.99	2.13	.000182	129	.0124	.0123	.274	131
						2.22	18.80	....	1.500	40.70	2.16	.000182	130	.0123	.0122	.256	133
						2.30	19.70	....	1.547	42.89	2.18	.000182	130	.0124	.0123	.269	132
						2.42	20.70	....	1.599	45.83	2.21	.000182	129	.0125	.0124	.278	131
						2.43	20.90	....	1.607	46.58	2.23	.000182	131	.0125	.0123	.264	131
						2.44	20.90	....	1.610	46.58	2.23	.000182	130	.0125	.0124	.271	131
						2.46	20.70	....	1.621	46.34	2.24	.000182	131	.0124	.0123	.263	132
						2.48	21.30	....	1.627	48.09	2.26	.000182	131	.0124	.0123	.261	132
						2.73	23.70	....	1.740	53.50	2.26	.000182	127	.0129	.0128	.321	127
62*a*	P, 1896....	......	......	Aqueduct of the Sorrino, Italy..	....	....	....	....	1.410	....	4.06	.00050	152	.0107	.0103	.042	150
63*a*	F-S, 1880...	......	......	Sudbury conduit, Boston water works; plaster of pure cement over hard brick.	490	3.071	....	....	1.863	....	2.529	.000161	146	.0114	.0113	.109	145
						3.574	....	....	2.048	....	2.672	.000160	148	.0114	.0114	.096	145
						3.768	....	....	2.111	....	2.805	.000158	154	.0111	.0109	.035	150
64*a*	C..........	1	63	Conduit, Cambridge, Mass......	....	1.75	....	....	....	....	....	.00025	....	.0110	....	....	....
65*a*	H, 1896.....	1	108	North Metropolitan sewerage system; East Boston pumping station; cement-washed brick trunk sewer.	....	1.02	3.96	6.39	.619	6.10	1.58	.000333	110	.0122	.0125	.340	122
		2				1.52	7.11	7.66	.928	15.67	2.21	.000333	126	.0117	.0117	.245	133
		3				2.04	10.85	8.98	1.208	29.40	2.70	.000333	134	.0116	.0114	.187	137
		4				2.45	14.00	9.95	1.408	42.18	3.03	.000333	139	.0115	.0112	.159	140
		5				3.16	19.92	10.88	1.830	69.50	3.48	.000333	141	.0117	.0117	.159	136
		6				3.75	25.12	12.57	1.999	94.60	3.73	.000333	145	.0116	.0115	.128	138
		7				4.62	32.86	14.24	2.309	138.00	4.18	.000333	150	.0115	.0113	.069	141
66*a*	H, 1897.....	1	108	Same reach of pipe, 16 months later.	....	2.15	11.67	9.12	1.280	30.13	2.55	.000333	123	.0126	.0125	.313	125
		2				2.74	16.42	10.53	1.560	47.75	2.90	.000333	127	.0127	.0126	.298	126
		3				3.19	20.15	11.44	1.762	62.05	3.06	.000333	126	.0129	.0129	.328	123
		4				3.20	20.22	11.42	1.771	64.82	3.18	.000333	131	.0126	.0125	.272	127
67*a*	H, 1900.....	1	108	Same reach after another lapse of 29 months.	....	1.99	10.44	9.32	1.120	24.96	2.38	.000333	123	.0124	.0123	.295	127
		2				2.83	17.10	10.66	1.606	48.26	2.82	.000333	121	.0132	.0132	.383	120
		3				3.64	25.08	12.85	1.952	76.78	3.16	.000333	124	.0133	.0134	.379	119
		4				4.18	28.89	13.57	2.130	95.84	3.30	.000333	124	.0136	.0136	.398	117

[1] In these tests the nominal or constructed slope of the pipe was accepted as the hydraulic grade. This probably does not introduce any error of moment, with the possible exception of observation 5 of pipe No. 54*a*, where the pipe was full, that is, under more or less pressure, which would have supported water columns at either ends of the reach tested, to such elevations that some other than the nominal slope might have been indicated, especially as the reach was relatively short, being but 800 feet long.

TABLE 11.—*Elements of experiments on partially filled concrete pipes and conduits for the determination of the retardation factors in various formulas*—Con.

1	2	3	4	5	6	7	8	9	10	11	12	13	Coefficients of retardation.				
													14	15	16	17	18
Pipe No.	Experimenter and year.	Observation.	Inside diameter.	Name and description of pipe.	Length of reach tested.	Maximum depth.	Area of water section.	Wetted perimeter.	Hydraulic mean radius.	Discharge per second.	Mean velocity per second.	Hydraulic grade per foot.	Chezy.	Kutter.	Manning.	Bazin (1897).	Williams-Hazen.
			d		*L*		*A*	*P*	*R*	*Q*	*V*	*s*	*C*	*n*	*n*	*m*	C_w
			In.		*Feet.*	*Feet.*	*Sq.ft.*	*Feet.*	*Feet.*	*Cu.ft.*	*Feet.*	*Feet.*					
68*a*	H, 1896.....	1	......	North Metropolitan sewerage system, below Charleston pumping station; section, basket-handle shaped, 6 feet by 6 feet 8 inches; 10 months in use.	5,000	1.02	4.32	6.28	.688	8.60	1.99	.0005	107	.0129	.0132	.392	116
		2				1.44	6.75	6.98	.958	16.59	2.46	.0005	112	.0131	.0133	.394	116
		3				1.91	9.50	8.01	1.187	26.81	2.82	.0005	115	.0132	.0134	.395	116
		4				2.40	12.40	8.95	1.387	38.82	3.13	.0005	118	.0133	.0133	.395	118
		5				2.89	15.40	10.00	1.539	52.90	3.44	.0005	124	.0130	.0130	.338	121
69*a*	H, 1897.....	1	......	Same reach 16 months later.....	....	2.91	15.00	9.76	1.540	45.67	2.97	.0005	107	.0149	.0151	.589	104
		2				3.29	17.90	10.90	1.650	56.67	3.16	.0005	110	.0147	.0148	.557	106
70*a*	H, 1900.....	1	......	Same reach; after another lapse of 29 months.	....	2.29	11.60	8.65	1.342	30.82	2.66	.0005	102	.0151	.0154	.620	102
		2				2.78	14.50	9.60	1.508	41.39	2.86	.0005	104	.0152	.0155	.633	102
		3				3.26	17.40	10.60	1.645	52.96	3.04	.0005	106	.0152	.0154	.626	103
71*a*	M, 1915.....	2	......	Cut-and-cover section, Catskill Aqueduct, Esopus division, stations 770+09 to 834+72.	6,463	4.54	71.80	23.90	3.000	261.40	3.64	.00021	145	.0123	.0123	.151	134
		1				5.40	86.60	25.70	3.370	346.50	3.99	.00021	150	.0121	.0121	.096	134
		3	......	Stations 131+57 to 254+34.......	12,277	4.35	68.40	23.60	2.900	261.40	3.82	.00024	144	.0123	.0123	.161	133
		2				5.22	83.40	25.30	3.290	346.50	4.15	.00024	147	.0123	.0123	.131	134
		4				6.50	105.50	27.90	3.780	468.70	4.44	.00024	147	.0125	.0126	.140	131
		1				6.43	104.30	27.70	3.770	471.80	4.53	.00024	151	.0122	.0123	.085	135
		3	......	Stations 131+57 to 208+65.......	7,708	8.00	131.00	30.90	4.240	625.00	4.77	.00024	149	.0125	.0126	.119	131
		2				7.85	128.50	30.60	4.200	621.90	4.84	.00024	152	.0123	.0124	.008	134
		1				8.78	143.90	32.50	4.430	736.50	5.12	.00024	157	.0120	.0120	.008	137
		1	......	Stations 131+57 to 178+17.......	4,660	9.26	151.70	33.50	4.530	756.50	4.98	.00024	151	.0125	.0125	.093	132

DESCRIPTIONS OF PIPES.

The following descriptions apply to experiments conducted by the writer for the Bureau of Public Roads. The pipes were laid on a hydraulic gradient and in no case were they flowing full of water. For descriptions of experiments conducted under similar conditions by other agencies see Appendix, page 77.

No. 49, Experiment S-37.—10-inch jointed concrete pipe chute-drop on L–Y–7 line, Tieton project, United States Reclamation Service, Washington.—Some of the topography of the Tieton project is such that water for irrigation must be conveyed down very steep slopes from one open channel to another. For this purpose chute drops of both the open type and the pipe type are used. A series of experiments was conducted on one of the pipes.

Water from the upper canal is delivered through the bank into a pool just above a 4-foot Cipolletti weir. After the water falls over this measuring weir it drops down a 16-inch intake well about 2 feet deep. At the bottom of this well the pipe chute, 10 inches in diameter, leads down the hill; falling 87.6 feet in a developed distance of 935 feet.

The volume of water, Q, was determined by hook-gauge measurements of head on the 4-foot weir, where the contraction conditions were good and the velocity of approach reduced by a brush screen. All of the items in Table 11 are based on the general hydraulic assumption that $A=\frac{Q}{V}\cdot$ As we have determined Q and V, we can solve for A as for a segment of 10-inch pipe. The resulting retardation factors for various formulas are all right in so far as they might be used in computing the velocity down a similar chute, but may not be used in the computation of maximum capacity for the following reason. All complete experiments upon friction losses in chute drops of either the open-channel or pipe type have developed the fact that the measured area of the wet cross section is much greater than $\frac{Q}{V}\cdot$ By a "complete experiment" is meant one in which Q, V, and the wet cross section, which we will call A', were measured independently. Such measurements disclose the fact that $Q=(A'-A'')V$ where A'' is an area made up of the aggregate entrained air bubbles. As a concrete example take observation 6 (see Table 11, p. 68). The velocity, V, as measured was 13.59 feet per second. The quantity, Q, as measured over the weir, was 4.29 second-feet. What might be termed the net water area $A=\frac{Q}{V}$ was 0.316 square foot. The nominal area of a 10-inch pipe is 0.545 square foot, but as most small pipes run under size the actual area was probably nearer 0.5 square foot. Thus, while by computation but 54 per cent of the cross section of the pipe was filled, yet the intake pool was full and no more water could be crowded into the chute. In other words, the pipe was filled with a mixture of air and water. At the outlet of pipe chutes running to capacity this fact is manifest by periodic rushes of air into the outlet pool.

It is not feasible and perhaps not possible to determine the mean area of the water section down a pipe chute by actual measurements. If such a thing were possible, then retardation factors could be computed, from which other similar chutes could be designed from a capacity standpoint without regard to the actual velocity attained. However, we do know, from experiments made on open-channel concrete chutes, that the value of n in the Kutter formula must be raised 0.002 to 0.004 when a chute is designed from a capacity standpoint.[1] In other words, the series of experiments on this 10-inch pipe chute indicates that a value of n of about 0.014 or 0.015 should be used in designing for capacity, instead of the values as shown in Table 11, namely, about 0.0119.

A view of one of these pipe chutes, in course of construction, and showing the vertical curve mentioned above, is given on Plate XXVIII in Volume II of Prof. Etcheverry's work. The writer believes this view is of the 10-inch chute discussed above.

No. 52, Experiment S-41.—36-inch jointed concrete flow line, Colorado Power Co., near Nederland, Colo.—In 1909 a flow-line conduit 12 miles in length was laid down the canyon of Middle Boulder Creek from the dam at Barker Reservoir to a small reservoir about 4 miles from Boulder, Colo., and about 2,000 feet in elevation above the creek bottom, where one of the plants of the Colorado Power Co. is located. This conduit is laid on an even gradient of 5 feet per 1,000 feet of pipe, except where inverted siphons are required to cross gulches and draws leading into the canyon. An article describing this installation says:[2]

> The 3-inch shell of the pipe that is laid on the hydraulic grade consists of one part Portland cement and three parts aggregate, graded from sand to stones having a maximum dimension of 1 inch. The pipe was cast in sections 2 feet long, with socket on one end and a bevel on the other end of each section to form the joints. Each section contains two hoops of No. 5 steel wire having a high tensile strength, one hoop being placed 6 inches from each end. These hoops are not considered to be of much value as reinforcing, but rather as an aid in preventing breakage before the concrete has set. The line of the conduit is very crooked, the longest tangent not exceeding 500 feet. Straight and beveled pipe were employed together to build the conduit on curves. The latter were located so their radii were as nearly uniform as possible, requiring only two kinds of beveled pipe.

The mixture was placed in rather dry layers 3 inches thick and carefully tamped. This allowed the forms to be pulled immediately after which the pipe section was kept thoroughly wet for 7 days, then

> the inner surface of the the sections was coated with a thick wash of neat cement to fill all irregularities. This surface was thus finished quite smooth, and, with the manner in which the joints were made, produced a conduit having a large carrying capacity.

During the season of 1915 the writer conducted a series of experiments upon the carrying capacity of a portion of this conduit near the upper end. A reach 1,977 feet long was chosen. Open standpipes near each end of the reach and approximately each 500 feet between the ends gave opportunity for measurement of the pipe segment not occupied by the water at five evenly spaced stations on the reach. For each run the quantity of water flowing in the pipe was varied by regulation of the outlet gates of Barker Reservoir. The velocity of the water in the flow line was determined by accepting the mean velocity of four batches of fluorescein for each observation. The color was injected at the standpipe marking the upper end of the reach and observed at a similar standpipe marking the lower end of the reach. The slope of the water surface was determined by piezometer tubes of type A connected with gauge glasses outside the pipe. The piezometer tubes were under identical dynamic conditions and examination of column 13, Table 11, shows that the corrected observed slopes practically agree with the nominal slope of the conduit. It was necessary to correct the observed slope for changes in velocity head between the upper and lower ends of the reach, the flow not being uniform throughout the reach tested. It is the writer's opinion, based upon his experience, that "uniform flow" is an ideal that is assumed in design but seldom attained in practice.

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[1] The fact that the fundamental hydraulic equation $Q=A\ V$ does not hold for chute drops, where high velocities, wave action, and turbulence exist, was first called to the attention of the writer by Mr. W. G. Steward, of the United States Reclamation Service. The results of his experiments, which have been corroborated in essentials by those of the writer, are found in Irrigation Practice and Engineering, by B. A. Etcheverry, Vol. III, p. 261.

[2] Engin. Rec., Nov. 6, 1909, vol. 60, p. 514.

The pipe was designed under an assumed value of n in the Kutter formula of 0.012 and observations made by the writer prove this assumption to have been correct, even after a period of 6 years without cleaning. So far as examination of the interior could be made from the various manholes the conduit is clean and practically free from slime. As the water comes from a large reservoir located on a mountain stream, it is clear and cold at all seasons of the year.

No. 56, Experiment S-49.—42-inch jointed reinforced concrete pipe, Victoria Aqueduct, Vancouver Island, British Columbia, Canada.—As mentioned under the descriptions of Nos. 30, 54*a*, and 55*a*, the Victoria Aqueduct consists of about 27 miles of flow line, broken by six inverted siphons (Pl. V, fig. 3, is typical of this flow line). Simultaneously with the experiments conducted on siphon No. 1 (pipe No. 30, p. 39) readings were also taken on gauges at manholes 3 and 4. This reach of pipe, 1,986 feet long, is downstream from the reach 800 feet long tested by Ehle in 1915 (No. 54*a*). Piezometer tubes of type A, under identical dynamic conditions, were held upstream against the current at the two manholes. True siphon tubes were carried over the edge of the manholes and connected the piezometers with graduated gauge glasses. The gauge glasses were then considered in a scheme of levels and the fall of the water surface was thus determined. This method is probably more accurate than to accept the nominal slope of the pipe. The areas of the water sections at the ends of the reach for the various runs were determined by careful measurements in the manholes. The discharge of the pipe was taken as the velocity in feet per second (determined by color tests in siphon No. 1), multiplied by the mean area of the siphon interior (p. 41). This discharge, divided by the mean area of the water section in the flow line, gave the velocity within the flow line. The reach tested was typical of the whole aqueduct, being about half curve and half tangent. The friction factors confirm those found by Ehle and show the same decrease in the values of n as the depth of water (consequently the velocity and the hydraulic radius for the depths considered) is increased. It was not feasible at the time these tests were made to turn sufficient water into the pipe line to fill completely the flow section, as a repair, due to a hillside slip several miles downstream, was in progress. The values of the retardation factors show that 0.011 is probably as low a value of n as is feasible to obtain in a commercial pipe and should only be used for pipes under practically ideal conditions, such as hold on this aqueduct.

CONCLUSIONS.[1]

From the facts developed in this investigation the following conclusions appear to be warranted.

In considering the design of a concrete pipe or conduit, from a capacity standpoint, the fundamentals to be kept in mind are:

(1) Original interior surface.
(2) Engineering; inspection and supervision of construction.
(3) The water.
(4) Acquired interior surface.

In a jointed pipe, the original interior surface depends upon that of the individual units and of the assembled whole. The smoothest units are cast of a wet mixture and allowed to set in rigid, smooth, oiled forms. These units are true to shape and assemble into a nearly perfect pipe. As the joints are practically as smooth as the cast surface, the number of joints is almost immaterial.

Most processes that permit immediate removal of the forms do not protect the plastic concrete against distortion. Distortion in the units means offsets in the pipe that may easily decrease the capacity from 10 to 20 per cent. This percentage may be reduced after assembling the units by tapering down the offsets with mortar applied with a trowel.

Supervision of the finished joints will usually improve the capacity materially. The good results obtained with the band method (see p. 85) appear perfectly feasible.

A brush coat applied to a rough original interior will increase the capacity and decrease the percolation. The same coat applied to a smooth interior will decrease its capacity and is not needed, under ordinary pressures, to prevent percolation, if the pipe is properly made. This is true for the reason that methods used in making pipe with a smooth interior are also conducive to a dense concrete. Note the suggestion that coating may scale off if radically different mixtures are employed for pipe and coat. (See p. 99.)

Pipe units made with the molds now on the market usually run undersize by as much as 3 or 4 per cent (in area) for diameters under 20 inches. Above that size, the nominal and actual areas are more nearly equal. Most pipes made with a wet mixture in short units, from which the molds are immediately stripped, "slump" to a slight extent, making a section somewhat undersize in area with a slight excess in shell thickness.

From a capacity standpoint monolithic pipes and conduits are built under two general conditions: (1) In the open or in the comparative freedom of a trench; (2) in a tunnel. Under the first condition the form adjustment, bracing, supervision, concreting, spading, and final inspection can be conducted from both inside and outside, with the added advantage of working in the daylight. Naturally this tends to result in better workmanship and higher carrying capacity than is feasible in a tunnel where all of the above operations must be done from the inside alone under the shadows and imperfections of artificial lighting.

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[1] These conclusions were written since receipt of the discussions beginning on p. 92, and are, therefore, based on all the data in the paper.

For all shapes, circular or otherwise, the invert is usually laid first. Forms for the remainder of the section are then placed on the invert and the section completed. Sometimes the first pouring extends just across the bottom and sometimes is carried up to the spring line. This depends on shape and size of the section. Smooth joints between the pourings and between the ends of adjoining set-ups are difficult to attain. Unless very heavily braced, the pressure of the head of wet concrete will spring the forms and result in offsets. On the Catskill aqueduct this amounted to as much as 0.15 foot.[1] The writer has seen cases where contractions of from 2 to 4 inches were made in sections 5 feet or less in diameter. Other factors that influence this movement are weather, length of sections, rapidity of filling the forms, and the type of aqueduct.[1]

Before the advent of steel forms, engineers anticipated a rough interior when the forms were stripped and relied on a plaster coat to fill interstices, estimates of cost being based on that of a plaster coat on a certain sized conduit equated against that of an unplastered surface on a larger conduit.[2] The present practice in general is to use smooth steel forms; clean them thoroughly after each casting, then dry and oil or grease them before using again. If close attention is given to spading the mixture into close contact with the forms, then a surface is secured that is smoother than will be attained by adding a plaster coat, local roughness alone being improved by pointing up and chipping down. The crown in particular is liable to be rough.[1]

Experience indicates that a given surface is more assuredly repeated in a pipe of precast sections than of monolithic construction. Thus a smaller factor of safety in the way of capacity overload may be assumed for precast pipe. The experiments indicate that some of the smoothest pipe is of careful monolithic construction; likewise some of the roughest. Thus, while a very smooth surface may be attained, it can not be anticipated with the assuredness possible where units are precast under the best practice and subject to the conditions attainable in a pipe yard.

For all types of pipe, competent engineering supervision and inspection are necessary if the best results are to be obtained. Velocity and entry heads should be provided at the intake. Alignment and grades should be true. Changes in either should be made with as gentle curves as are feasible. Unless it is excluded above the intakes, means should be provided for the removal of débris at the foot of steep *upward* slopes. Means for the removal of air, not only at the summits but also near the intake (unless deeply submerged) should be provided. This is true of small orchard lines as well as large siphons. Trash racks, originally placed at the intakes of many siphon pipes, have since been removed in many cases because the danger from washout, due to the accumulation of trash, outweighed the good accomplished.

For all practical purposes it does not appear necessary to limit the higher velocities. The velocity should be at least sufficient to prevent the deposition of silt—probably not lower than 3 feet per second.

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[1] Water Works Handbook, by Flinn, Weston and Bogert, New York, 1916, p. 276.

[2] Jour. West. Soc. Engrs., Vol. XVI, No. 8, 1911, p. 696. Discussion of C. C. Saner.

More or less slime may be expected from waters stored in reservoirs containing algæ growths or from waters that have been conveyed long distances in openearth channels. (See Mr. Hazen's discussion on p. 97). This may also be true, but to a less extent, if the open channel is of wood or concrete. Waters of the Southwest are less liable to develop retarding growths in covered conduits than those of the East and Northwest. Water containing calcium carbonate ($CaCO_3$) will smoothly coat pipe of any material. If sand, gravel, or other débris is also present in the water, the cemented accretion will be much more harmful, being rough and more rapid.

After construction a rough pipe may become somewhat smoother if conditions are such that a mineral coating is deposited. An originally smooth surface will remain about as constructed or depreciate slightly by mineral or slime deposits. Slimes come quickly or not at all. A depreciation of from 10 to 15 per cent in capacity can occur within five to seven months, but does not greatly increase after that. Thus, to be of material benefit, cleaning must be frequent if necessary at all. For this reason periodic cleanings have been abandoned in many places.[1] If algæ slimes are anticipated, it may be best to allow for 15 per cent depreciation of capacity and no cleaning.

A concrete surface is not subject to a progressive roughening influence like the tuberculation found in iron and steel pipes, though tuberculation of the reinforcement may occur if too near the surface. Roots may affect a poorly made or poorly laid pipe, but will not influence a hard, well-jointed pipe.

In considering various kinds of pipes it is not sufficient to compare concrete with wood, concrete with steel or iron, concrete with vitrified clay, or even concrete with concrete. This is true not only from a capacity standpoint, but also from the standpoint of strength, life, and operation. One particular class of concrete pipe must be compared with the pipe of the other materials or with concrete pipes of the other classes. Two estimates or bids for a concrete pipe may differ 20 per cent, yet from a capacity standpoint alone the higher in cost may be the more economical.

ACKNOWLEDGMENTS.

The writer desires to acknowledge indebtedness to the various engineers and managers of irrigation, municipal, and power systems who permitted and aided in tests upon the pipes in their charge; also to the Board of Water Supply of New York City and to the Ontario Power Commission of Toronto, Canada, for supplying hitherto unpublished results of tests made on their conduits.

Tests made on pipes of the various United States Reclamation Service projects were made under a cooperative agreement between that service and this bureau. Especial acknowledgment is due the officers and men of the Reclamation Service who were unfailingly courteous, energetic in getting the best feasible conditions for experimentation, and in many cases devoted to these tests long hours outside their regular work.

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[1] Water Works Handbook, Flinn, Weston and Bogert, New York, 1916, pp. 290–291.

APPENDIX.

The following pages are devoted to abstracts of the descriptions of experiments made by agencies other than the Irrigation Division, Bureau of Public Roads. The first part covers tests made on pipes under pressure while the last portion covers tests made on pipes and conduits but partially filled. The number before each description refers to the corresponding numbers in column 1, Tables 3, 4, and 11.

PRESSURE PIPES.

No. 12, Experiment N-1.—16-inch jointed concrete pipe, Z_a, siphon, Umatilla project, United States Reclamation Service, Oregon.—In 1911 and 1912 Herbert D. Newell, manager of the Umatilla project, conducted a series of experiments on several of the concrete inverted siphons under his charge. For the sake of brevity the descriptive matter pertaining to all the tests will be abstracted from his extended article,[1] and placed under this number. Matter pertaining to individual pipes will be placed under the proper reference number. He states:

> During 1911 and 1912 a number of experiments were made to determine the coefficient of friction. The quantity of water was generally determined by meter measurements. Difference of water surface elevation between the inlet and outlet ends were determined from bench marks carefully established on the inlet and outlet structures.

Regarding the 16-inch pipes he adds:

> No information exists as to whether or not the 16-inch pipes are somewhat obstructed near the bottoms of depressions. It will be noted that the 16-inch pipe show discharges relatively much smaller than the 30 and 46 inch pipe. The 16-inch are made by the dry process and have joints every 2 feet. At every joint there is unavoidably a slight irregularity in the cross section. The mixture used in making the 30 and 46 inch pipe is distinctly a wet mixture. All sizes are grouted on the inside, but the grouting on the large size is more smooth, as a man can work inside the pipe and it is possible to do a better job.

The nominal size of the pipe appears to have been accepted as the true size. In the experiment on this particular siphon the quantity of water for the first observation was taken as the mean of two meter measurements, one indicating a discharge of 3.78 second-feet and the other 3.70 second-feet. For the second observation the mean of three measurements, ranging from 4.78 to 4.97 second-feet was accepted. From the amount of the friction loss the writer would judge that some débris had accumulated between the time this pipe was laid and the date of the experiments. The retardation factors are not consistent.

No. 13, Experiment N-2.—16-inch jointed concrete pipe, Z_c siphon, Umatilla project, United States Reclamation Service, Oregon.—In addition to the information given under reference number 12*a* for the series of tests conducted by Newell, the following pertains to this pipe alone:

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[1] Studies of Coefficient of Friction in Reinforced-Concrete Pipe, Umatilla project, Oregon. By H. D. Newell, Eng. News, vol. 69, May 1, 1913, p. 904.

The quantity was taken as the mean indicated by two current meter tests, one giving 3.19 and one giving 3.65 second-feet. This agreement is not as close as it should be. For this reason the writer would not give too much weight to the results of the experiments on this pipe in considering the friction loss which appears rather low for this type of pipe. The pressure head was measured by a "testing gage." The one observation on this pipe shows a value of $C_s = 0.314$.

No. 19, Experiment.—Fanning 20-inch cement-lined wrought-iron force main pipe.—In 1880, J. T. Fanning conducted a series of experiments and listed the results in his well-known work on hydraulics,[1] but he does not give a description of the pipe or of his methods of experimentation. Hering and Trautwine say of this pipe:[2]

> No short bends, but two large Y branches, two small blow-off branches, and three stop valves.

No. 24, Experiment N-3.—30-inch reinforced concrete jointed pipe, D_1 siphon, Umatilla project, United States Reclamation Service, Oregon.—This pipe, laid in the winter of 1909–10, was tested for capacity by Mr. Newell in 1911 and by the writer in 1915. (See No. 23.) For general data on the Newell tests see No. 12*a*.

The elevations of the surface of the water at the inlet and outlet structures were determined by the use of a hose pipe and water pail, in a manner similar to that later used by the writer when other methods were not feasible. By means of a true siphon over the wing walls the surface of the water in the pail and in the canal are brought to the same level.

For observation No. 1 a single weir measurement was accepted as indicating the discharge, while for observation No. 2 a single meter measurement indicated a discharge of 16.62 second-feet and a measurement at the weir at the inlet indicated a discharge of 17.04 second-feet. As noted in Table 2 the various elements have been computed for both these indicated discharges, rather than averaging them. This method follows Mr. Newell's original article.

A comparison of the platted points for this pipe (Pl. VI) indicated that it was very smooth when new, but had become somewhat obstructed when tested by the writer four years after Mr. Newell's tests. The relationship of the leading canal to the intake clearly shows that this is highly probable. The average value of C_s for the three observations equals 0.408, which is higher than usual, even for the best of construction.

No. 25, Experiment N-4.—30-inch reinforced concrete jointed pipe, R_3 siphon, Umatilla project, United States Reclamation Service, Oregon.—This pipe was tested by Mr. Newell when it was in its fourth season of service. The discharge was taken as the mean of three measurements by current meter, ranging from 12.76 to 12.42 second-feet, to which was added 0.032 second-foot which passed a weir. Mr Newell states: "It is not unlikely that there is a considerable deposit of sand in the first low depression." This would not appear to the writer to be a serious deposit, as the friction loss indicates a very efficient pipe. The relatively great length of the reach, 3,658 feet, makes this a valuable test, so far as one observation is indicative. The value of C_s equals 0.351.

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[1] A Practical Treatise on Hydraulic and Water Supply Engineering, by J. T. Fanning, 11th ed., New York, 1893, p. 238.

[2] E. Ganguillet and W. R. Kutter, translated by Rudolph Hering and John C. Trautwine, jr. A General Formula for the Uniform Flow of Water in Rivers and Other Channels. New York, 1907, 2d ed., p. 154–155.

For additional general information which also applies to this pipe see No. 12*a*.

No. 27, Experiment B-1.—31.5-inch (0.8 meter) experimental conduit, Dijon, France.[1]—In 1895, M. Henry Bazin conducted a series of experiments upon "a cement pipe, 0.80 meter (31.5 inches) in diameter and 80 meters (262 feet) long. This pipe was straight, perfect in bore, and opened at its extremities into two basins, 2 meters wide and 15 meters long, having vertical walls."

The velocities in the pipe were measured with a pitot tube, operated in one of three shafts, which divided the pipe into four equal parts. Each of these shafts was the full width of the pipe, 0.8 meter, and 0.8 meter in length, measured along the axis of the pipe. The discharge was also measured over a sharp-edged rectangular weir 2.01 meters wide, without end contractions, located 50 meters beyond the lower basin. The coefficient of discharge of this weir had been previously established.

The loss of head was determined over a reach of 40 meters (131 feet) between the two end shafts. "On the right of each shaft was installed a manometer, consisting of a glass tube with a scale, the former being connected with the interior of the pipe by an orifice 0.002 meter in diameter, pierced in the wall, without any protuberance." Considering the shortness of the reach, reference to Plate VI shows a remarkably consistent set of observation points, indicating an exponent of V equal to 1.971 and values of C_s equal to about 0.418. If this value of C_s truly represents the condition of this conduit, then a surface smoother than that of the Victoria Aqueduct is indicated, even considering the curvature of the latter. To the writer this is almost inconceivable, in commercial construction.

No. 33, Experiment N-5.—46-inch reinforced concrete jointed pipe, R_1 siphon, Umatilla project, United States Reclamation Service, Oregon.—This pipe, laid in the winter of 1909–10 was tested in 1911 and again in 1912 by Mr. Newell and in 1915 by the writer. (See No. 32.) In 1911 the elevations of the water surface in the inlet and outlet structures were determined by the use of hose and pail, mentioned under No. 24*a*, and in 1912 a "testing gauge" was used. A close agreement resulted between the two methods of gauging. The experiments by both Mr. Newell and the writer indicate that this is a very efficient pipe. Although not so smooth as the Victoria Aqueduct the absence of curvature gives about the same friction factors. For general information that also applies to this pipe see No. 12*a*.

No. 36, Experiment of Budau.—7.22 foot reinforced concrete pipe, Perlmoos cement works, Sell-Leukenthal, Austria.[2]—Prof. A. Budau conducted a series of tests upon a new pipe line serving a power plant. This conduit was 4,200 feet long and 7.22 feet inside diameter. No mention is made in the elaborate description of the tests as to whether the pipe is jointed or monolithic, made over wood or steel forms, coated or as left by the forms, exactly the nominal size or otherwise. The line has several curves in horizontal alignment, but each one covers such a small angle that from the standpoint of capacity the pipe may be considered straight. In profile the pipe is on a continuous gentle down grade. Loss of head was measured with water columns in graduated glass tubes, installed at each end of the reach tested. The relative elevations of these gauges were established by the static head; that is, with still water in the line. The piezometer connections were of type D (fig. 3), different from anything used in American experiments, so far as the writer is informed. A perforated brass ball, 30 millimeters in diameter, was set at the center of the pipe. In order to annul all influence of velocity head and secure only pressure head, the balls were filled with brass shavings. The usual tubing connection is made between the balls and the gauge glasses, through stuffing boxes set in the shell of the pipe.

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[1] Trans. Amer. Soc. Civ. Engin., vol. 47 (1902), p. 246.

[2] Experiments on pressure losses in iron reinforced concrete conduits, by Prof. A. Budau, from the "Zeitschrift des Osterr. Ingenieur und Architekten-Vereinnes No. 8," Feb. 20, 1914, p. 141.

The volume of water was measured over a specially formed weir, having end contractions.

Piezometer No. 1 was located about 1½ diameters from the intake of the pipe line. This would appear to be at a point subject to much disturbance. The weir is located about 8 or 10 feet below the power house and no mention is made of baffles to still the velocity of approach. These conditions may offer grounds for criticism of this series, although the platted points appear to be consistent with the exception of observation No. 3, which was excluded by the writer in consideration of this series, as there is probably some error of observation or typography.

No. 38, Experiment M-1.—110-inch siphons, Catskill Aqueduct, New York.—The Catskill Aqueduct contains several pipe siphons using identical construction for inlet and outlet chambers and for the siphon proper. The latter is in essentials a steel pipe, lined with 2 inches of cement mortar and protected outside with at least 6 inches of concrete.[1]

One siphon was lined with a cement-gun process while the others were grouted. For both processes, the invert, for a width of about 8 feet of arc, was lined in the manner employed in laying sidewalks. When the cement gun was used the coat was applied in layers in rapid succession. Each layer is sufficiently rough for a secure bond with the next one. The resulting surface was floated and troweled to secure a smooth finish. The cement-gun process was abandoned by the contractor after one siphon had been so lined. In the grouting method, metal-covered wood forms were used after extensive experimentation. The grout was placed through a 2½-inch pouring pipe inserted in a rivet-passing hole and subjected to a head of about 4 feet above the zenith of the pipe. The first batch or two of grout for each set-up, 15 feet in length, was mixed 1 part cement to 1 of sand) while the succeeding batches were mixed in a ratio of 1 to 2. By both processes "very smooth interior surfaces were secured." (See p. 82.)

In Table 3 are shown average coefficients for observations on the Esopus, Tongore, Foundry Brook, Sprout Brook, Peeksill, Hunters Brook, Turkey Mountain, and Harlem Railroad siphons, for a flow of 350 million gallons per day (542 second-feet). Concerning these observations, made in August of 1915 under the direction of Mr. F. F. Moore, the board of water supply writes:

> The condition was not favorable for obtaining the best results on the pipe siphons. * * * The result of measurements of flow in pipe siphons was recorded as an

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[1] Engin. Rec., vol. 63, Apr. 15, 1911, p. 404; *id.*, vol. 64, Sept. 16, 1911, p. 332; Eng. News, vol. 66, p. 526; Municipal Jour. and Engineer., Vol. 31, Aug. 23, 1911, p. 229: *id.*, Dec. 7, vol. 31, 1911, p. 719: Jour. N. E. Water-Wks. Assn., Sept., 1911: Waterworks Handbook, by Flynn, Weston, and Bogert, New York, 1916, p. 323.

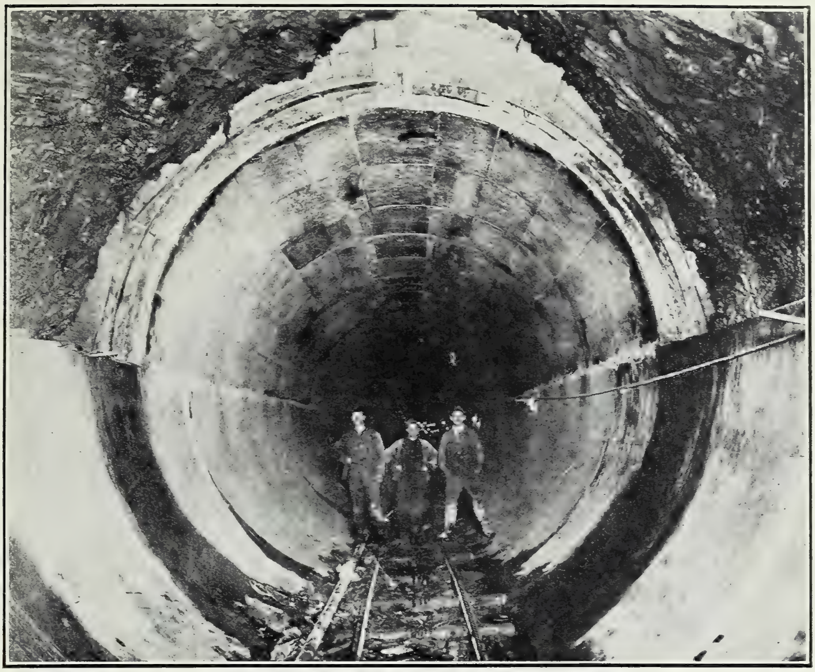

FIG. 1.—RONDOUT TUNNEL, CATSKILL AQUEDUCT.

Typical pressure tunnel construction, showing completed excavation and partial and completed concrete lining.

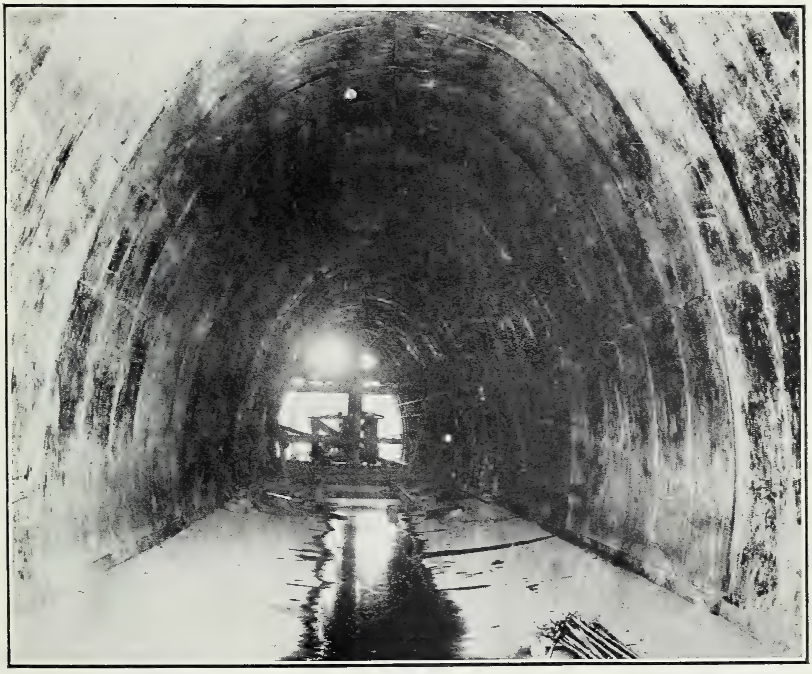

FIG. 2.—CUT-AND-COVER SECTION OF CATSKILL AQUEDUCT, PEEKSKILL DIVISION.

Interior of finished conduit on curve of 3,000 feet radius.

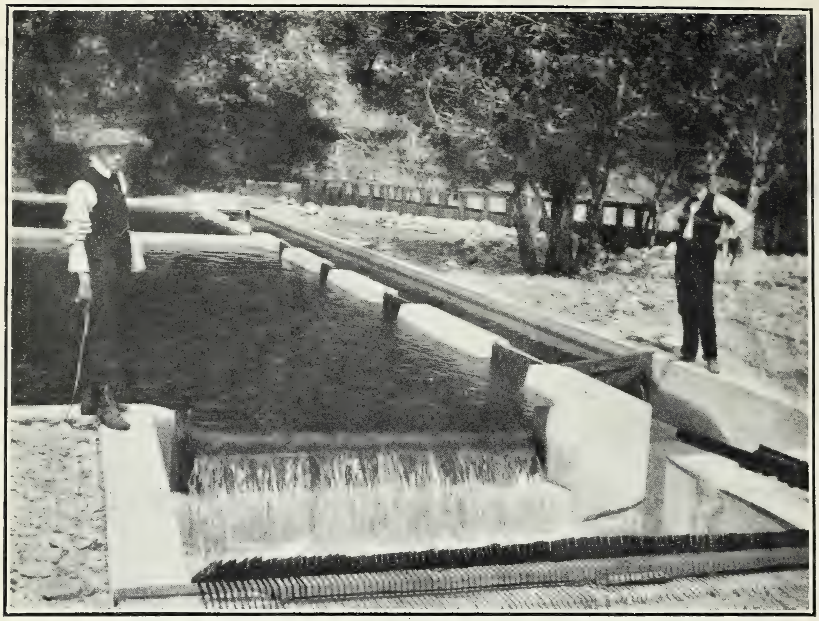

FIG. I.—WEIR AT INTAKE TO MILL CREEK NO. 3 PIPE LINE, CALIFORNIA.

Used in measuring the water entering pipe No. 51a.

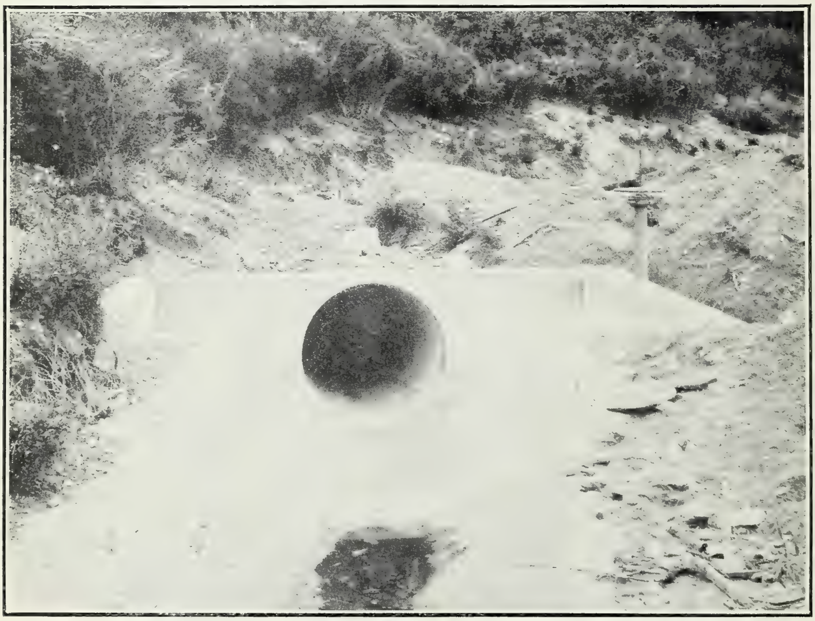

FIG. 2.—INTAKE, LYTLE CREEK POWER PLANT PIPE LINE, CALIFORNIA.

Tapers from 44 inches diameter to 36 inches at pipe line proper. Sand trap shown between open canal and pipe line. (No. 53a.)

average, because observations were not of a character permittting reduction for individual pipes. These pipe measurements included the chamber and transition-section losses, of all kinds, at both ends of each pipe because it was not considered advisable at that time to provide piezometers, or other devices, necessary to the direct measurement of the pipe losses separately. These observations were reduced by taking advantage fo the fact that the pipes were of various lengths and the same type of construction, including the chambers, so that the assumption could be reasonably made that the wetted surfaces are practically identical in character: hydraulically, and that chamber losses are the same in all chambers for any given flow conditions. Losses in the pipes were computed, using assumed pipe friction-loss coefficients, which computed pipe losses were subtracted from the measured losses. These computed residual losses were than compared and the set most nearly consistent selected. The coefficient used in obtaining the selected set was assumed to be an average for the several pipes and an approximately correct average because of the method used and the probability that not all pipes were in the same condition as to foulness, hydraulically considered. The value selected is believed to quite closely approximate the truth, because some of the siphon pipes are so short that the total friction loss in them is very small as compared to the chamber losses.

While the observations are not such that definite conclusions therefrom would be warranted, still the indications are that a steel pipe, built up of relatively long sections, well jointed and lined with a smooth cement coating, gives a very efficient surface. Of course the primary object of this coat is to prevent the corrosion of the metal interior but is gives an added satisfaction to know that this prevention has been attained without sacrifice of capacity, for a given sectional area. If anything, the fractional loss is less than it would have been in a new metal pipe and the latter material would have continually lessened in capacity while the lining will probably remain about the same, after the first slime coat is acquired.

No. 39, Experiment M-2.—Rondout pressure tunnel,[1] Catskill Aqueduct, N. Y.—At the Rondout River crossing the Catskill Aqueduct takes the form of a circular pressure tunnel, excavated in solid rock and lined with concrete. From the standpoint of capacity the tunnel is, in essentials, a circular pipe, constructed in place. (See Pl. IX, fig. 1.)

The tunnel consists of vertical downtake and uptake shafts joined by an approximately horizontal tunnel. The developed length of the tunnel is 24,880 feet. About 16,000 feet from the intake a vertical drainage shafts extends from the ground line down to the tunnel. As the ground line at Rondout River is 300 feet below the hydraulic gradient, this shaft is, of course, sealed.

In August, 1915, F. F. Moore, designing engineer of the board of water supply, conducted a series of experiments to determine the friction losses in this tunnel, from the drainage shaft to the outlet, a distance of 9,102 feet. The pressure head at the drainage shaft was measured with a mercury manometer of the pot-and-column type. Mercury readings were corrected for temperature of air and of water in the tunnel. The temperature of the water in the pipes to which the manometers were attached was assumed to be controlled by the ground and therefore unchanging. The mercury column was also read with no flow in the tunnel, thus establishing the levels between gauges.

The elevation of the water surface at the outlet was determined by steel-tape measurements from a bench on the floor over the uptake shaft. Disturbance was eliminated at this point by passing the flow over stop planks below the shaft.

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[1] Eng. Rec., Mar. 11, vol. 63, 1911, p. 279; Eng. News, June 1, vol. 65, 1911, p. 654; Water Works Handbook, by Flynn, Weston, and Bogert, New York, 1916, p. 284.

In the computations it was assumed that the velocity head within the uptake shaft was changed to pressure head without loss. Observations 1 to 4, inclusive, were computed from 31 readings at 5-minute intervals for each observation. For the other observations readings were taken at minute and half-minute intervals for 124 readings. Observations were made about noon on each day, after which the discharge was changed and allowed until the next noon to secure a stable regimen of flow. As stop planks caused a break in the hydraulic gradient immediately below the reach tested, this regimen of flow was quickly attained.

The discharge was measured through a 210-inch Venturi meter located 8 miles above the tunnel. The leakage was known to be small and negligible from tests previously made.

At the time of test the tunnel was new, but had been in operation for sufficient time to allow the surface to become somewhat foul, although the observations were not continued long enough to indicate the probable ultimate deterioration, hydraulically considered, of the surface which long and constant use might bring about.

The concrete lining, 17 inches in thickness, was deposited against clean, oiled steel forms, being carefully spaded near the forms to assure a dense, smooth inner surface. The mixture used was, in general, composed of one part cement to two of sand and four of crushed stone.[1]

In conducting the experiments, the discharge (hence the velocity) was reduced from day to day until five observations had been taken. It was then increased from day to day until three more observations had been recorded, the velocities being about the same as those for the first three observations.

A study of Plate VI indicates that for a given loss of head the velocity during the latter end of the series would be about 13 per cent less than that occuring during the beginning of the series.

In reply to a question as to "shoulders" within the tunnels, the board writes:

> Regarding "shoulders," by which is understood the irregularities in the concrete surface of waterways occurring at the places where form sections joined, there were such irregularities as commonly occur in the pressure-tunnel linings, but this effect hydraulically is unknown, because of lack of opportunity to compare, under otherwise identical conditions, the measurements as made with results which might be obtained with a smoother lining. The mortar linings of the pipe siphons were unusually free from form irregularities, such as exist being clearly of negligible effect, hydraulically.

No. 40, Experiment M-3.—Wallkill pressure tunnel, Catskill Aqueduct, N. Y.—At the Wallkill River crossing the Catskill Aqueduct takes the form of a circular pressure tunnel, 4.4 miles long, excavated in solid rock with a minimum cover of 150 feet, the tunnel being then lined with concrete. Thus, like the Rondout tunnel, the conduit is a circular pipe, when considered from the standpoint of capacity. (See Pl. IX, fig. 1.)

The concrete was placed as a moderately dry mixture of 1 part cement to 6 parts of aggregate.[2] Careful spading against the steel forms, which were well oiled or greased at each set-up, assured a dense, smooth surface.

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[1] Engin. Rec., Jan. 1, vol. 61, 1910, p. 26; *id.*, Jan. 29, vol. 61, 1910, p. 138; *id.*, Feb. 28, vol. 69, 1914, p. 240; *id.*, Sept. 17, 1910, vol. 62, p. 312.

[2] Eng. Rec., Apr. 2, vol. 61, 1910, p. 450.

The equipment and method of taking the observations were the same as those employed in the tests of the Rondout tunnel (p. 81).

It is to be noted from Table 3 that the same discharge, hence the same velocity, held for the observation taken on this tunnel and the Rondout tunnel, for any given day. The tunnels are 5 miles apart, and this one is 18 miles beyond the Venturi meter, where the discharge was determined. (See p. 82.)

When the position and sequence of points is studied on Plate VI the position does not appear to follow a rather definite relationship to the sequence, as was the case for the Rondout tunnel, but the points appear to be indiscriminately placed on either side of the line through the centers of gravity, as is the case with nearly all hydraulic experimentation.

No. 41, Experiment RDJ.—18-foot monolithic concrete-lined tunnel, No. 2 conduit of Ontario Power Co., Niagara Falls, Ontario, Canada.—For the following data concerning some unpublished experiments the writer is indebted to the Hydro-Electric Power Commission of Ontario. These experiments, conducted under the supervision of R. D. Johnston, are of especial importance for two reasons. So far as the writer is aware, the conduit is the largest on this continent, if not in the world. The velocities encountered in commercial operation are three or four times as great as those ordinarily considered as high velocities in long penstocks. The following description is submitted by the commission:

> This conduit consists of approximately 6,726 feet of concrete pipe, of which 6,646 feet, located between the gatehouse and surge tank, was tested. Of this 6,646 feet approximately 5,170 feet is straight pipe and 1,475 feet in length is bends to the right of 800 feet radius. These bends consist of short curves separated by short tangents.
>
> The pipe is of oblate shape, approximately 18 feet in diameter, the greatest horizontal dimension being 19.26 feet and the greatest vertical 16.55 feet. The surface of pipe is unusually smooth and even. During construction great care was taken to see that the concrete was carefully spaded next to the oiled steel forms, and after the whole pipe had been erected all defects due to imperfect alignment of forms were removed by chipping and then the whole inside was rubbed down by hand with carborundum bricks. This pipe delivers water by means of a distributor and penstock to 7 turbines developing a total of approximately 91,500 horsepower. The pipe was put in operation in 1910 and the tests were made in 1913 and 1914. An examination of the interior made in April, 1918, showed no signs of cavitation or wear and a total absence of vegetable growth. This latter condition is probably due to the extremely high velocities, which at times reach 28 feet per second. The inside surface is shown on the accompanying photographs taken during the inspection mentioned above. (Pl. XI.)
>
> For the tests the velocity was measured by the "color method." This consists of liberating some coloring substance in the water at a known point and at a given time and recording the time taken for this color to be carried to another known point. The computations were based on an elapsed time, as from the moment of injection to the mean between first and last appearance of the color. The volume of the pipe or other container between the two points is measured or calculated, and this gives the total volume of water which has passed the second point of observation in the recorded time. The velocity of water at any section can then be calculated. In this case the point of introduction of the color was in the mouth of the pipe in the gatehouse and the point of observation was at the tailrace weir opposite No. 7 unit, which is the first one served by this conduit. Adjustments necessary for the change in velocity in the penstock, etc., were made after further tests had been made on the latter. (For a more detailed description of this method see Engineering News of Sept. 23, 1915.)
>
> The loss of head was obtained by observing the water levels at the gatehouse and at the surge tank. The surge-tank riser served as a huge manometer, in which all

small fluctuations were damped, so that close readings were made possible by means of an attached mercury gauge. The difference in level of the water at these two points, which was corrected, in the case of the surge-tank readings, to allow for the velocity head of the water which passed the surge-tank connection, included, of course, the loss at entry to the pipe line. The engineers who made the tests found it impossible on account of the disturbance to measure the pressure of the water in the conduit immediately behind the gatehouse, where the only manhole for a considerable distance is located.

Great care was exercised to obviate errors in observation or calculation.

To obtain the coefficient of the conduit and plot a curve of losses for different velocities of water involved a long series of observations and a large amount of computation. The coloring material used was potassium permanganate in the form of small crystals. It was advisable to use as little of the color as possible and still enough to enable the observer to distinguish it easily at the power-house weir. When the water was clear it required about two-thirds of a pint of crystals. Roily water required considerably more.

The apparent loss of head measured in the conduit is the elevation of water in the forebay minus the elevation of water in the surge tank. There are three corrections to be applied to this resultant figure. It is assumed that the elevation of water in the surge tank is lower than a pitot-tube reading in the conduit opposite the surge tank branch by an amount equal to the velocity head of the water passing the surge-tank branch. Hence this velocity head is added to the level of the water at the gatehouse to give the correct figure for entry loss plus conduit loss, plus distributor loss from gatehouse to surge-tank branch. The entry loss based on the judgment of the engineers and on data impossible to give herein was taken as one-quarter of the velocity head or $\left(\frac{1}{4}\times\frac{V^2}{2g}\right)$. The distributor loss from unit No. 7 to the surge-tank branch was obtained by experiment and checked by computation. These two quantities, namely, entry loss and distributor loss, were deducted from the total loss and the net loss in the conduit, due to friction and curves, obtained. By this method about 34 runs were made and computed during the summer of 1913 and 8 more during the summer of 1914. The total range of velocity was from 7 feet per second to 21 feet per second.

The items under No. 41*a*, in Table 3, are not computed from the 42 runs mentioned in the last paragraph above, but are developed from a velocity-friction loss curve which is itself based on those 42 runs. From this curve the velocities and corresponding friction losses were chosen and the remaining hydraulic elements and the retardation factors in the various formulas computed.

A study of these retardation factors indicates that this conduit is exceptionally smooth. Only in cases where conservation of head is of great value in dollars and cents would the expense of hand treatment, such as is described here, be warranted. If we take the coefficients of retardation at their face value, then this pipe will carry from 10 to 15 per cent more than an average pipe of the type that the writer has classified as of the highest commercial construction. However, it is to be borne in mind that the experiments on this conduit determined gross losses of head from which loss by friction alone must be developed under certain assumptions. While no criticism of the assumptions is made, still the fact remains that, had it been feasible to determine friction losses alone, it is quite possible that the seeming variation of 10 to 15 per cent mentioned above might have been much reduced.

FLOW-LINE PIPES AND CONDUITS.

No. 50, Experiment FF-1.—22-inch jointed concrete pipe of Southern California Edison Co., Mill Creek power plant, No. 2, California.—The description of the experiment upon this line, as taken from correspondence with Mr. Finkle reads:

The pipe was made in 2-foot sections in the ordinary plain cement-pipe forms as used for that purpose, and consisted of 1 part Portland cement and 3 parts sand mixed

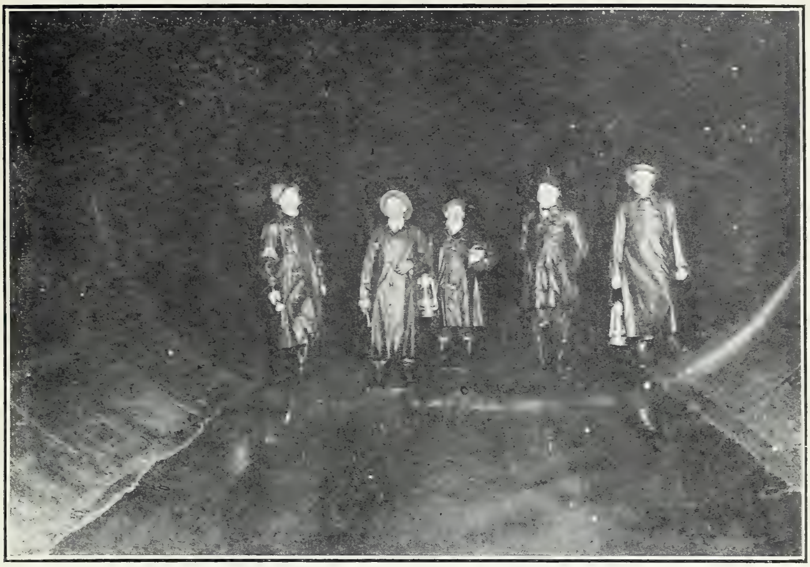

FIG. I.—INTERIOR, CONDUIT NO. 2, ONTARIO POWER CO., NIAGARA FALLS, ONTARIO.

Note from text that form scars were chipped down; then whole surface rubbed down with carborundum bricks.

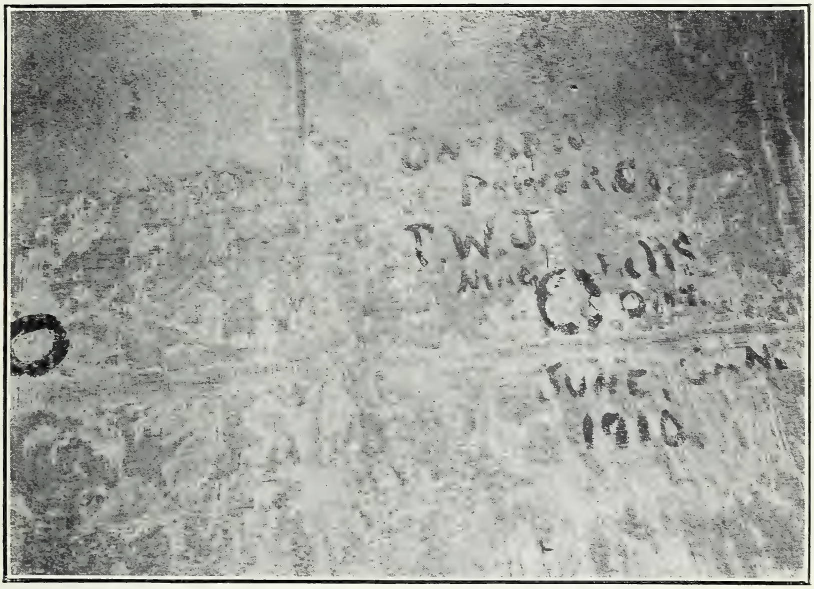

FIG. 2.—MINIMUM OF EROSION BY CLEAR WATER.

After eight years' service the conduit shown in figure 1 has not been eroded sufficiently to obliterate paint marks made on walls in 1910. As velocities reach 28 feet per second, this is remarkable evidence that good concrete does not erode under sliding contact with clear water. This view taken March 31, 1918.

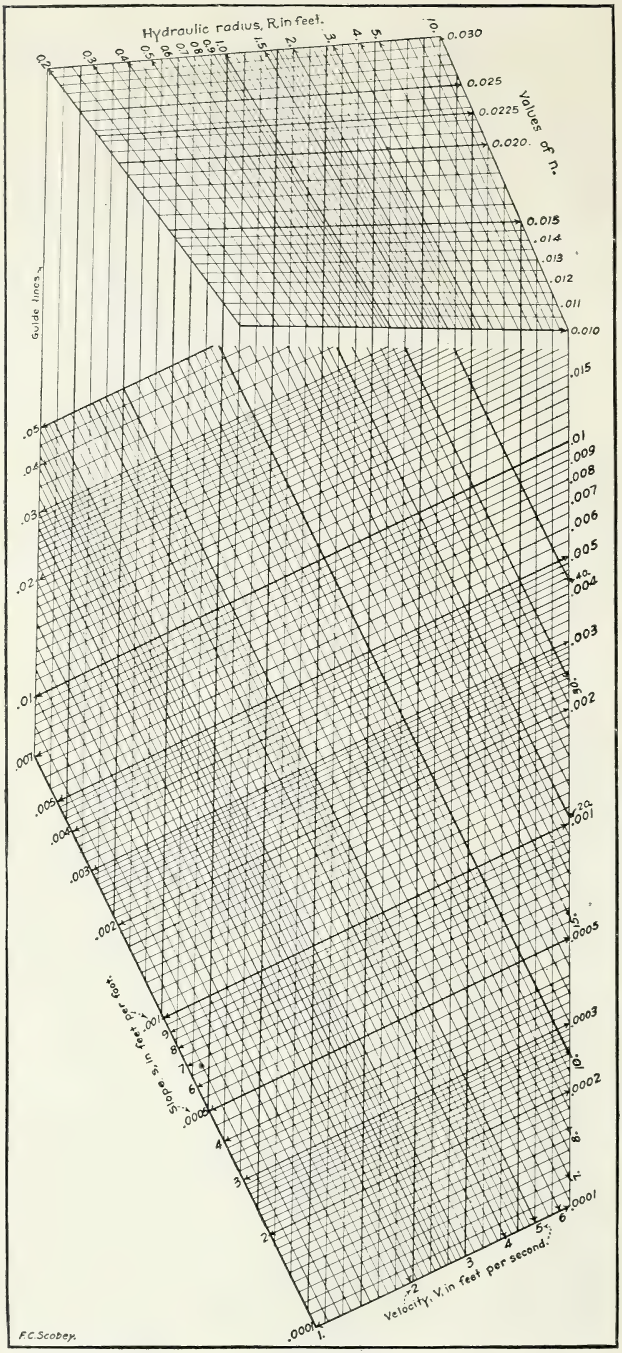

DIAGRAM FOR THE SOLUTION OF GENERAL PROBLEMS INVOLVING THE KUTTER FORMULA.

From the intersection of R and n follow the guide lines to the intersection of s and V, or from the intersection of s and V follow the guide lines to the intersection of R and n. For circular pipes remember that D=4 R.

with fine gravel not exceeding 1 inch in diameter. The sections were joined together in the ordinary way with cement mortar mixed with 1 part cement to 2 parts fine sand. The line has tangents ranging in length from about 100 feet to 500 or 600 feet, and curves, none of which have a less radius than 15 feet. The slope was made uniform at the rate of 10.56 feet per mile.

This pipe line delivered at its lower end, when filled to its full capacity, 9.2 second-feet without any pressure at the intake, which was provided with an enlarged section for accelerating the flow of water in the pipe. It was observed that the pipe did not run full below the first 2,000 feet. Subsequently the upper 100 feet of the pipe were raised to accelerate the water a little more, after which numerous tests were made to determine the greatest volume of water that could be passed through the pipe. From this I found that the pipe would carry the most water when filled within 1 inch of the top, or when carrying a depth of 21 inches of water. Under this condition the pipe delivered 9.8 second-feet of water. The discharge of the pipe was measured by means of a rectangular, fully contracted Francis type of weir in one-eighth inch steel plate, and head taken with hook gauge. The depth of water in the pipe, from which the velocity and hydraulic radius are computed, was measured by means of a straightedge and hook gauge at the manholes, which are located every 500 feet along the line. Instead of using a trowel to make the joints, as is ordinarily done, I made a brass band ring 10 inches wide, having spokes equipped with turnbuckles on the inside to vary its diameters, the ring being flexible and lapped to permit reducing or enlarging its circumference.

In making the joints the pipes were carefully laid and evenly joined, and mortar applied in the groove all around. Then the brass ring, reduced in diameter to slip into place, was introduced and centered over the joint. By working the turnbuckles the ring was then expanded to fit the diameter of the pipe and its inside circumference, thus squeezing the mortar into the crack and the surplus out to the edges of the ring.

Grasping the spokes, the ring was then turned around slowly about five times to give the joint a smooth surface, after which it was loosened and removed, and all surplus mortar removed with a trowel, in such a manner as not to mar the surface left by the ring.

This method made the joint smoother than any other portion of the pipe, and, so far as the eye could detect, made the pipe continuous on the inside. Had the same method been used for the 31-inch pipe for Mill Creek No. 3 line (No. 51) and for the 36-inch pipe for Lytle Creek (No. 53) I have no doubt these would have given about the same value for n in Kutter's formula.

The nominal slope was used in computation.

As shown in Table 11 the above test indicates the value of n to be about 0.0116 for this pipe. If truly representing the inner surface of the pipe, the joints were most carefully made and the pipe in excellent condition. (See discussion No. 51 following.)

No. 51, Experiment FF-2.—31-inch jointed cement pipe of Southern California Edison Co. Mill Creek power plant, No. 3, California.—The description of the experiment upon this line, as taken from correspondence with Mr. Finkle, reads:

The diameter of this pipe was 31 inches and it was given a slope of 10.56 feet per mile. Its length was between 5 and 6 miles, with a few interruptions in the line, where steel siphons were used to cross ravines. Having profited by my experience regarding accelerations at the intake, showing that the pipe would be full at the upper end and only partly filled at the lower end, I gave this line additional fall in its upper part, calculated by a formula for that purpose.

This pipe was manufactured in the same way as the 22-inch pipe (No. 50, p. 84), except that 1 part of cement was used and 4 parts of sand, containing considerable gravel, the largest of which was 1½ inches in diameter. It was made in 2-foot sections. and laid in the same manner as the 22-inch pipe. The result was that the maximum capacity of the pipe occurred when it was filled to within 1½ inches of the top, or when it only carried a depth of water of 29½ inches. The carrying capacity of the pipe under these conditions was 19.72 second-feet.

This line was laid in 1901 and tested the same year. There are numerous curves between the tangents of various lengths, but none of the curves has a radius of less than 20 feet. The discharge was determined by using a Francis type rectangular, fully contracted weir in three-sixteenths-inch steel plate, measuring head with a hook gauge. (See Pl. X, fig. 1.)

The depths of water in the pipe, from which the velocity and hydraulic radius are computed, were measured by placing straightedge along inside of upper invert on each side of manholes and using hook gauge from top of straightedge down to surface of water.

The nominal slope of the pipe was taken in the computations of the retardation factors for the various formulas. As shown in Table 11, page 68, this experiment indicates a value of *n* in the Kutter formula of 0.0142, and so far as can be determined from the descriptions of this pipe and the 22-inch pipe the values are accounted for by the manner of making the joints. (See discussion No. 50 above.)

No. 53, Experiment FF-3.—36-inch jointed cement pipe of Southern California Edison Co., Lytle power plant, California.—The description of the experiments upon this line, as taken from correspondence with Mr. Finkle, reads:

> This 36-inch line, laid in 1904, was given a slope of 5.28 feet per mile, and allowance made for acceleration of the water at the intake (as explained for pipes Nos. 50 and 51, which see also for manufacture of pipes). The pipe showed a delivery of water amounting to 20.06 second-feet, when running within 1½ inches of the top, or with a depth of water equal to 34½ inches in the pipe.
>
> In the year 1908 a second test showed the capacity had increased, so as to deliver 21.2 second-feet under the same conditions as it previously delivered 20.06 second-feet. Investigation showed that this was due to a fine deposit of carbonate of lime formed on the interior of the pipe, almost making a glazed surface over the cement. This was deposited by the water of Lytle Creek, which contains considerable lime carried in the water in the form of bicarbonate.
>
> This line contains numerous curves between tangents of various lengths, but none of the curves has a radius less than 34 feet.
>
> The discharge was determined by passing water over a rectangular, fully contracted Francis-type weir in three-sixteenths-inch steel plate, and depth was observed with a hook gauge. The immediate intake was tapered from 44 to 36 inches in diameter (see Pl. X, fig. 2). The depths of water in the pipe, from which the velocity and hydraulic radius are computed, were measured by holding a straightedge against the upper invert on each side of the manholes and measuring from the top of straightedge in manholes to the surface of the water.

The nominal slope of the pipe was taken in computing the retardation factors for the various formulas. According to the experiments, the glazing action of the lime improved the value of *n* from 0.0145 when the pipe was new to 0.0138 when it was 4 years old. Experiments at the end of, say, 10 years might show a very efficient surface but a decrease in capacity, due to throttling of the water section, assuming the deposition of lime to continue.

No. 54, Experiment Ehle-1—Victoria Aqueduct, Vancouver Island, British Columbia, Canada.—Water was first turned into this pipe line on May 14, 1915 (see Nos. 30 and 31). On May 23 and 24, 1915, Boyd Ehle conducted a series of tests to determine the friction factor *n* in the Kutter formula.[1] As shown in Table 11, p. 68, the range of these tests extended from 3.36 second-feet flowing with a depth of but 8⅝ inches up to 40.59 second-feet which filled the pipe, thus extending through the transition from an open channel (i. e., with water surface exposed to the air) to a pipe running full of water. It is not often that an installation is such that the behavior of water may be studied through this transition, as a pressure pipe is full of water, whatever the discharge, and a flow-line pipe as a rule is protected against complete filling by spillways near the upper end of the line.

The reach covered by this series was 800 feet long. The upper end is near the pipe-line intake at Sooke Lake. The discharge was measured over an 8.02-foot sharp-crested rectangular weir, corrected for velocity of approach. The depth of water upstream from the weir crest was 3.40 feet and the end contractions 1.50 feet and 1.55 feet, respectively. For all discharges less than 17.0 second-feet, the bottom and end contractions were in excess of two times the head on the weir. For greater discharges than the above, the end contractions are not sufficient to give standard conditions. The mean velocity for any particular run of water was taken as the discharge divided by the mean of the areas of the water sections at the two ends of the reach. The slope was taken as the constructed slope of the pipe line, a fall of 1 foot per 1,000 feet of pipe.[1]

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[1] Engin. Rec., Oct. 2, 1915, vol. 72, p. 409.

The reach tested consisted of about equal amounts of tangent and curve, as is typical of this line, but did not include any of the siphon pipes. There is so much curvature on this line (50 per cent) and the individual curves are as a rule so sharp that the values of the coefficients of retardation may be taken as for a very smooth pipe with an excess of curvature. In other words, the average pipe with the same type of construction would probably show even more favorable carrying qualities.

No. 55, Experiment Ehle-2.—42-inch reinforced concrete flow line, Victoria Aqueduct, Vancouver Island, British Columbia, Canada.—With the following modifications the same discussion applies to this reach as to No. 54 above:

After flowing for 27 miles through a flow-line pipe, except for six inverted siphons, laid on a uniform grade of 1 foot fall per 1,000 feet of pipe, the water wells up through 110 holes 4 inches square from an outlet chamber just upstream from an 8.04-foot weir similar in construction to the one at the intake, except that the depth from the perforated floor to the crest of the weir is 1.88 feet and the end contractions are 5.5 feet each. For all observations the end contractions were more than twice the head on the weir; but the bottom contraction did not conform to standard conditions for a discharge greater than 17 second-feet.

This series of observations was made on a reach of pipe about 800 feet long, just upstream from the weir described above where the discharge was measured.

No. 58, Experiment JBL-1.—Tunnel No. 15, San Gabriel plant, Pacific Light & Power Co., California.—Just before starting the construction of the Los Angeles Aqueduct a series of experiments was made on both open an covered concrete channels, located in southern California. The measurements, made by Charles H. Lee and D. L. Reaburn, were reported by J. B. Lippincott[2] under whose direction the experiments were conducted. (See p. 11 for Mr. Lippincott's conclusions.) Of this tunnel he writes:

> The section is rectangular, 4½ feet wide and 4 feet deep, with a semicircular arch, and finished with a 1 to 3 cement-mortar plaster.
>
> There was no vegetable growth in the tunnel which could be felt or seen. The tunnel has been in use for eight years. There is a slight curve at the upper portal. Discharge measurement was made on tangent above the curve. Twenty feet beyond lower portal there was a sharp angle in alignment.

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[1] In the opinion of the writer the slope should have been taken as the fall of the surface of the water in the pipe, corrected for changes in the velocity head due to changes in the areas of the water sections at the two ends of the reach. However, his own experience on this same pipe indicates that practically the constructed value for the slope is obtained when determined by the surface fall with the above corrections. (See column 13, Table 11, opposite pipe No. 56.)

[2] Observations to determine the value os C and n as used in the Kutter formula by J. B. Lippincott, Engin. News, June 6, 1907, vol. 57, p. 612.

The discharge was measured with a current meter. Depths were determined from a horizontal straightedge. On account of rough topography, floor elevations as furnished by the company were accepted in computing the slope of the water surface. The value of *n* was found to be 0.0127. (See p. 11 for Mr. Lippincott's conclusions.)

No. 59, Experiment JBL-2.—Tunnel No. 23 of the same system as above.—Of this experiment Mr. Lippincott writes:[1]

> This test was made at tunnel No. 23 of the same system. Conditions were similar in all respects, except for slight curves just below north portal and above south portal. The length of tunnel is 318 feet and grade of floor 0.00095 foot per foot; *n* here was 0.0115.

(See p. 11 for Mr. Lippincott's conclusions.)

No. 60, Experiments JBL-5 and 6.—Main supply conduit for Los Angeles, Calif.—These tests are described as follows:

> These were on the main supply conduit for Los Angeles, near the old ostrich farm, several miles north of Los Angeles. It is covered and has been in use four years. The length of section was 700 feet, being between two manholes. A curve occurred on this section. The wetted perimeter was very smooth, apparently of a 1 to 3 cement mortar. There was no sand, moss, or vegetable growth of any kind. Experiment 5 was at the upper and 6 at the lower manhole. The grade of the floor was the same as that of the water surface. The surface was a cement plaster on concrete. The value of *n* here was 0.0112 and 0.0109. (See p. 11.)

No. 61, Experiment AFB-1.—Loch Katrine Aqueduct, Glasgow, Scotland.—The concrete-lined portion of the new aqueduct from Loch Katrine to Glasgow was tested in 1895 by A. F. Bruce.[2] Of the construction he writes:

> Open frames of 6-inch by 2-inch battens were first placed in position and three-fourth-inch tongued and grooved boards, smeared with soft soap, nailed to the frames horizontally as the concrete was filled in. Every possible precaution was taken by working with spades to obtain a good face, and except where some defects showed themselves, no redressing was afterwards necessary.

About 53 per cent of the aqueduct was lined. The section units were from 12 to 15 feet long, generally 12 feet. The reach tested was straight. The quantity of water was measured over a weir. The depths of water in the aqueduct were read on gauge rods in the chambers. Water at the lower end was checked by the fact that one siphon gate below the reach under test was closed.

The lined section is 9 feet 1 inch wide on the bottom, with the invert dished 6 inches. The sides batter until at the spring line the width is 10 feet. The rise of the arch is 3 feet. Depth over all is 9 feet.

Most of the observations indicate a correct value of *n* for this new channel of about 0.0124.

No. 62, Experiment P-1.—Aqueduct of the Serrino, Naples, Italy.—In 1896 Perrone wrote[3] of a test made on the aqueduct of the Serrino, at Naples, Italy. This channel, of pure cement, polished, had vertical sides and elliptical bottom. The discharge was measured with a current meter. The value of *n*, being but 0.0107, indicates the workmanship and lack of slime to be all that the short description implies.

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[1] Observations to determine the value of *C* and *n* as used in the Kutter formula by J. B. Lippincott, Engin. News, June 6, 1907, vol. 57, p. 612.

[2] Observation on the Flow of Water in the New Aqueduct from Loch Katrine; Glasgow Corporation Waterworks, A. F. Bruce, Paper No. 2921, Pro. Inst. Civil Engineers, Vol. CXXIII, 1895–96, part 1, p. 410.

[3] Zoppi, Sul Volturno, Carte Hydrographique d'Italia; The Flow of Water, by Louis Schmeer, New York 1909, pp. 46, 80, 93.

No. 63, Experiment F-S.—Sudbury Aqueduct, Metropolitan water works, Boston, Mass.—Alphonse Fteley and F. P. Stearns conducted a series of experiments upon the Sudbury Aqueduct in 1880.[1] The conduit was new, of horseshoe shape, 9 feet wide and 7.7 feet high. The discharge was measured over a weir. This aqueduct contained sections lined with brick alone, and also sections lined with a coating of cement mortar over the brick. After stating that the capacity of the brick section is represented by the formula $V=127R^{0.62}s^{0.5}$, they add:

> When the inside of the conduit is lined with a coating of mortar made of fine Portland cement, its flowing capacity is from 7 to 8 per cent greater. This coating, though applied with floats, did not present as smooth a surface as was obtained in other portions of the conduit, where experiments would probably have given higher results. In some parts of the conduit where the brick surface was covered with a wash of Portland cement laid with a brush, the flowing capacity was increased to the extent of from 1 to 3 per cent.

Note that the exponents in the above formula are the same as those adopted by the writer. Also see discussion by Mr. Hazen on page 97. For the cement-lined section their formula would have read $V=137 R^{0.62}s^{0.5}$, which becomes, in terms used by the writer (see p. 49), $V=0.394d^{0.62}H^{0.5}$, indicating a capacity about 6 per cent greater than the formula for the best grade of construction as suggested by the writer.

Nos. 65, 66, and 67, Experiment H.—108-inch cement-washed brick sewer, North Metropolitan sewerage system, Boston, Mass.[2]—Theodore Horton describes experiments conducted upon a 9-foot circular brick sewer, the interior of which had been washed with cement mortar. So far as carrying capacity is concerned, the pipe is of cement. Extending over a period of several years, these tests show what may be expected with the lapse of time, when sewage is conveyed in a concrete or cement lined channel. Below the East Boston pumping station the cross section of the sewer is a 9-foot circle for 2,000 feet; thence a horseshoe shape of the same area as the circle for 2,000 feet; thence another 9-foot circle for an additional 3,000 feet. A uniform gradient of 1 foot fall in 3,000 feet is maintained. The exact length of reach tested is not disclosed, but the statement is made that simultaneous measurements of depth showed the flow line above the horseshoe cross section to be parallel to the invert. Thus the nominal grade of the line might be accepted without correction for any change in the velocity due to nonparallel flow. The discharge was determined with carefully conducted current-meter gaugings and the cross section by actual measurement. The first series was made in 1896, 10 months after the system at this point had been put in operation. The next series was made in 1897, after these channels had been in operation about 26 months, during which time ample opportunities for changes in the carrying capacity had taken place.

The third series was made in 1900. The sides of the sewer, above and below the average water line, were covered with a thin coating of grease of a leaden color, and supported an organic growth, probably of a fungus nature. By this time there were slight incrustations at frequent intervals, due to barely visible seepage of ground water, and the growth, while not much heavier, covered a greater area on the sides of the sewer. The incrustation seemed slightly heavier, but the organic growth did not show any marked increase over that at the time of the 1897 tests.

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[1] Trans. Amer. Soc. Civil. Engin. vol. 12, p. 17. See also Jour. Assoc. Eng. Socs. vol. 26, p. 163.

[2] Trans. Amer. Soc. Civ. Eng., 46 (1901), p. 78.

In his discussion Mr. Horton accepts the value of n of 0.0117 for 1896, of 0.0127 for 1897, and of 0.0133 for 1900. (See Table 11, p. 69 for individual values of each observation.) He notes that the deposits of grease and of organic matter were greatest on the sides, especially near the line of average flow; that the bottom was clean scoured by sand and other heavy particles.

In his discussion on these experiments, Rudolph Hering mentions several cases where the deposit exceeded that noted by Horton. Mr. G. C. Whipple brings out the point that the organisms which grow in sewers are vegetable forms of a very low type, while those found in water pipes belong to the animal kingdom—fresh-water sponges, etc.

The results of these tests would indicate that sewage undoubtedly coats a cement surface to such an extent that an exceptionally smooth interior surface would not long contribute to a high capacity, as would be the case for most clear waters. By the same deposits, on the other hand, a very rough interior might be so smoothed over that the capacity will be increased, provided the sewage was sufficiently diluted with surface or other ordinary waters, thus preventing excessive deposits.

Nos. 68, 69, and 70, Experiment H-2.—Basket-handle cement-washed brick sewer, North Metropolitan sewerage system, Boston, Mass.—Simultaneously with the tests described as Nos. 65, 66, and 67, Horton also made experiments upon the basket-handle section adjoining the circular section. The shape was 6 feet by 6⅔ feet, but otherwise similar in construction to the circular section. The general description under the latter applies also to this section.

The difference in the values of n for the two shapes would indicate that a better surface is obtained in a circular form than upon plane faces, or those requiring hand troweling. A review of tests for carrying capacity shows this fact to be quite general.

No. 71, Experiment M-4.—Cut-and-cover sections of the Esopus division, Catskill Aqueduct, New York State.—Long reaches of the Catskill Aqueduct were constructed as flow-line sections, as distinguished from the reaches under pressure.[1] These sections are of the cut-and-cover type, a concrete horseshoe 17½ feet wide and 17 feet high.

The invert, 16 inches thick, was laid first, in alternate panels each 15 feet long. After the forms had been filled—

> The work is screeded with a 16-foot length of 3-inch circular steel shafting, which is rolled along the top of the forms, bearing on the web plates and the horizontal legs of the angle irons riveted to them. Great care is taken to obtain a smooth finished surface.

For the walls and arch, steel forms in 5-foot units bolted into 15-foot sections were used. From invert to arch any one section was completed at one pouring. After the oiled forms were in position, four men crawled between the forms, two on each side. As the concrete, a wet mix in a ratio of about 1 to 3 to 5, was poured in from the top, these men spaded it away from the inside form, so that the inner surface of the conduit is very smooth. (See Pl. IX, fig. 2.)

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[1] Engin Rec. vol. 61 Jan. 8, 1910; vol. 62, Nov. 5, 1910; Water Works Handbook, Flinn, Weston and Bogert, New York, 1916, p. 271.

Simultaneously with the hydraulic experiments upon the tunnels and cement-lined pressure pipes described on pages 80 to 83, inclusive, measurements were made on three reaches of the cut-and-cover flow lines. For nearly half the runs nonparallel flow was indicated. That is to say, the slope of the water surface was either greater or less than the slope of the invert. The computations were not completed for such runs, but for all runs where the flow was uniform the hydraulic elements are given in Table 11, page 70. The quantity was measured by Venturi meter, as described for the pressure conduits. The areas and wetted perimeters of the water sections were determined by measurements from arch intrados to water surface. The velocities were computed from the continuity equation $V=\frac{Q}{A}$.

The coefficients of retardation indicate that quite high efficiencies are perfectly feasible in conduits of great size if care is taken to work the concrete mix into close contact with smooth forms and if care is exercised in moving the forms so that no offsets or shoulders are developed. A glance at the items in Table 11, page 70, shows the same indication toward incipient fouling that was found in the experiments on the pressure tunnels and pipes. The sequence of tests is shown by the order of observation numbers. For any given reach of conduit the retardation factors indicate a rougher surface from day to day when the water is first turned into a conduit.

DISCUSSION OF " FLOW OF WATER IN CONCRETE PIPE."[1]

By Kenneth Allen, *Sanitary Engineer, Board of Estimate and Apportionment, New York City;* Arthur S. Bent, *Engineering Contractor, Los Angeles, Calif.;* F. C. Finkle, *Consulting Engineer, Los Angeles, Calif.;* Allen Hazen, *Consulting Engineer, New York City;* J. B. Lippincott, *Consulting Engineer, Los Angeles, Calif.;* H. D. Newell, *Project Manager, Umatilla Project, United States Reclamation Service, Hermiston, Oreg.*

DISCUSSION BY MR. ALLEN.

It is fortunate that in these times of soaring prices for cast-iron pipe a competing material for many purposes is found in reinforced concrete, for a careful perusal of the author's important and exhaustive investigation demonstrates that if we can be sure of the best material and fabrication, this material can be safely used—at least under reasonable head—and with economy.

In estimates for a 48-inch force main 12,350 feet long, based upon prices ruling in the summer of 1916, there was found a saving of $84,000 by the adoption of precast concrete pipe instead of cast iron, or 44 per cent of the cost for the latter. The difference to-day (Feb. 16, 1918) would be still greater.

Precast concrete pipe appears to be particularly well adapted to sewer construction for several reasons: Joints are infrequent and consequently infiltration should be small; the width of trench is a minimum, causing less inconvenience to traffic and reducing the excavation; there will be little danger from corrosion due to acids sometimes observed with cast iron when used for sewers; and, finally, instead of a marked decrease in carrying capacity, due to tubercles, it would appear that the excellent surface obtained when first laid is subject to little deterioration from time. This means that for the same ultimate capacity we may assume no larger diameters and gradients, and possibly less, than for cast iron—certainly less than would be justified with ordinary brickwork or mass concrete. Therefore the best precast pipe would seem particularly adapted to the construction of those sewers where, on account of the flat topography or in order to reach the outlet at the desired elevation without pumping, minimum gradients must be employed.

With a sewer 4 feet in diameter, for instance, and with a minimum velocity when flowing full of 2½ feet per second, the frictional loss per mile will be with—

New cast-iron pipe=2.11 feet (Williams and Hazen formula C_w−130).
Old cast-iron pipe =5.17 feet (Williams and Hazen formula C_w=80).
Ordinary mass concrete =2.72 feet (author's formula C_s=0.31).
Best precast concrete =1.91 feet (author's formula C_s=0.37).

In other words, under the conditions assumed, the friction loss with cast-iron pipe will vary from 1.1 to 2.7 and with ordinary concrete work will amount to 1.4 times that to be had with the best precast concrete pipe.

With any concrete pipe subjected to pressure, as in the case of slabs, bins, or concrete ships, reliance must be placed on the excellence of the material and workmanship entering into the structure. Thorough inspection is quite as important as careful design and well-drawn specifications to avoid the possibility of failure.

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[1] The original manuscript of the preceding paper was submitted to the men named, who in their varied experiences are familiar with concrete pipes for the conveyance of water for irrigation, power, and municipal use, and also for the conveyance of sewage. Criticism and discussion of the manuscript were asked for. Acknowledgment is now made of the time and labor expended gratuitously in preparing the discussion and comments given here. Throughout the discussion "the writer" will refer to the name heading that particular part of the discussion and "the author" will refer to the author of the paper.

DISCUSSION BY MR. BENT.

The author has reached a conclusion that in an empirical and ignorant way I reached many years ago. Our own struggle in laying concrete pipe lines has been, as the result of these conclusions, to secure the greatest possible smoothness of joints. I think we began to realize the importance of this shortly before we built the 12-mile line in Boulder Canyon, which the author has mentioned, and it was very interesting to learn that after these years of service he found the line carrying the quantity of water for which it was figured. At the time we were laying it, the Central Colorado Power Co.'s engineers were in a very anxious state of mind because some one had accidentally used a lower value for n than the chiefs later on approved, but it was then too late to change the diameter of the pipe. It was freely predicted that there would be bitter disappointment in the amount of water delivered. We tried to have the disappointment on the right side and succeeded by care in the work.

The entire line was laid by us under contract and the joints inside and outside were made of 1:2 mortar put on solely with a trowel.

I used to talk about 0.012 being perfectly safe for n and 0.011 as being perfectly attainable. I still believe this was right, but it is so difficult to control field conditions that for some time we have pushed these values up one figure, and that seems to be about the conclusion the author has reached.

DISCUSSION BY MR. FINKLE.

From experience with the manufacture of concrete pipe and its use, the writer has come to the conclusion that the carrying capacity of such a pipe is almost entirely in the hands of the engineer having charge of its design and construction. This statement relates more to the ordinary gravity-flow concrete pipes made in 2-foot sections and joined in the trench, but it will also apply in a considerable degree to other classes of concrete and cement pipes.

Three points are important in attaining a high carrying capacity of concrete pipes. These may be briefly stated as follows:

First, use as wet a mixture as possible and thoroughly settle the concrete in the forms, making every effort to have gravel and crushed rock in the mixture covered by a film of the fine sand and cement mixture on the inside of the pipe. This object can be attained by using more forms, so the sections can remain longer in the forms and a wetter mixture can be employed. The increased cost of this will be slight, as it will only be the interest on investment and wear and tear of the additional forms.

Second, in laying the pipes the joints must be carefully made so that there will be no projections or rough places where the sections are united. This can be accomplished by using the revolving brass band described in connection with the author's No. 50, experiment FF 1.

Third, careful attention must be paid to the alignment and grade on which the pipe is laid. This is important, as it eliminates irregularities in flow due to uneven bottoms and angles or irregularities.

The writer's experience has been that curves should have a radius of not less than six times the diameter of the pipe.

If these precautions are taken there is no reason why concrete pipes can not be so produced as to have a value of $n = 0.011$, or even 0.0105 in Kutter's formula. It is therefore principally a question of engineering, which I am sorry to say has been greviously neglected in connection with concrete pipe work in the past.

Little light can be thrown on the possible carrying capacity of concrete pipes by studying most of the old pipes of this class found in southern California, as they were nearly all laid out without any engineering supervision, or a proper survey to give uniform grade and proper alignment. A review of the author's experiments on these old pipes clearly prove this point.

In practice, the carrying capacity of concrete pipes, in common with other kinds, is often greatly reduced by the entrance of air at the intake. It is often customary to take the water into a pipe line from the overfall of a Francis weir for measuring it. The effect of this is to cause a large volume of air to be carried into the pipe with the water. The writer has often found this condition in connection with concrete pipes in southern California and elsewhere, and, after correcting it, has increased the carrying capacity of the line from 10 to 20 per cent.

The examples of concrete pipe given by the author, with which the writer was connected, are the 36-inch Boulder Creek concrete pipe line in Colorado, designated No. 52, experiment S 41, for which work he prepared the specifications and acted as consulting engineer. The experiment from which the author obtains a value of $n = 0.012$ was near the upper end of this pipe. If the test had been made at a point farther down the line it is probable that a value of $n = 0.0116$, or better, would have been obtained, the same as for the Mill Creek No. 2 pipe (No. 50, experiment FF 1), because the two pipes were manufactured and laid in the same summer.

It has been well demonstrated as a hydraulic principle that there is a slight but constant acceleration of the velocity in long gravity-flow pipe lines of this kind, and the writer has found a difference in the value of n near the upper and lower ends of such pipes, where all the conditions were exactly alike. This phenomenon has been referred to by some authorities and called by the name "Constant acceleration in gravity conduits."

Both the Boulder pipe (No. 52, experiment S 41) and the Mill Creek No. 2 pipe (No. 50, experiment FF 1) were made and laid by day work under careful engineering supervision, which explains the results obtained as to carrying capacity, although these pipes were both made in 2-foot sections by the dry-mix process. It is the opinion of the writer that, had these pipes been made with wet mix, they would have showed a value of $n = 0.0105$, or probably even a little better.

As to the other two pipes, for which the writer was engineer, namely Mill Creek No. 3 (No. 51, experiment FF 2) and Lytle Creek (No. 53, experiment FF 3), it must be remembered that both of these were installed by contract and without proper precautions in regard to making the joints, which is wholly responsible for the higher value of n and less carrying capacity of these pipes.

From the above and from all other experiences of the writer, his conclusion is that the values of n given by the author, as applicable to the best constructed pipes, which can be produced by the dry-mix method, are as stated by the author, namely, for from 12 to 24 inch pipes $n = 0.011$; from 26 to 48 inch pipe $n = 0.0115$; and for pipes over 50 inches in diameter $n = 0.012$.

On the other hand, pipes made with a wet mix and laid with equal care by using the brass-band device, or some other method for making the joints smooth, should show better results, probably about as follows:

For pipes 12 to 24 inches in diameter, $n=0.010$ to $n=0.0105$.
For pipes 26 to 48 inches in diameter, $n=0.0105$ to $n=0.011$.
For pipes over 50 inches in diameter, $n=0.011$ to $n=0.0115$.

In reference to the decrease of carrying capacity in concrete pipes, after they are laid, the writer has formed the following opinions from his own personal observations:

First. There is no risk of interfering with the carrying capacity of concrete pipes on account of roots entering them if they are properly made and laid. There is no case of record where roots have entered cement pipes, unless they were made without being properly tamped, or the spaces in making field joints were not properly filled with mortar.

In this respect concrete pipes are different from vitrified-clay pipes, from which it is very difficult to exclude the roots of certain trees. This difference is due to the fact that it is not easy to cause proper adhesion between cement mortar and vitrified clay, when making field joints, while perfect adhesion between a concrete pipe section section and the mortar used for making joints can always be had, so as to eliminate all possibility of roots entering at the joints.

Second. Some concrete pipes have become deteriorated through scour, the surface having become pitted and rough. Several cases of this kind have been observed by the writer in southern California.

Such instances, however, are due to one of two things, either on account of too little cement in the concrete from which the pipes were made, or subjecting properly made pipes to unreasonable water velocities, particularly when the water carries sand or silt. Proper engineering will prevent anything of this kind, as every case where it has been done is an instance of design and construction without proper engineering advice.

Third. It is possible, under many conditions, to have accretions occur on the interior of concrete pipes, whereby their capacity will be decreased. The most frequent cause of this is the deposition of mineral carried by the water in solution, either in its pure form or combined with silt and sand, carried by the water in suspension.

As shown by the author, the most common mineral causing deposition on the interior of concrete pipes in southern California is bicarbonate of lime, and this will apply equally to any other locality where waters contain bicarbonate of lime in solution. Unless the water is very heavily impregnated with bicarbonate of lime and the velocity in the pipe is rapid, the deposition is very slow and would require a long time to make much change in the capacity of a pipe line, unless the water also carries such matter as silt, sand, and fine gravel in suspension.

The writer has observed that the deposition of bicarbonate of lime is much more rapid from water which is warm than from cold water. This observation was made in connection with steel and iron pipes in the domestic water system in the city of Rialto, San Bernardino County, Calif., which carried water from Lytle Creek. Attention was first called to the matter about 10 years ago, when it was discovered that a film of bicarbonate of lime had been deposited on the interior of the 36-inch concrete pipe line (No. 53, experiment FF 3) referred to in the author's treatise.

The writer then made an examination of the steel and iron pipes in the domestic water system at Rialto, carrying the same kind of water, and found these pipes coated in the same manner as the concrete pipe, the thickness of the accretion depending upon the length of time during which the pipe had been in service.

Some of these iron and steel pipes had been laid since the year 1885, and during the 20 years of their use had accumulated from one-fourth to three-eighths of an inch coating on the inside. After that two experimental steel pipes were installed for observing the effect of water. One of these was placed in position in the month of November, and when observed again the following April there was very little evidence of any deposit. At the same time, another pipe section was installed new, just below the one which had been in use from November to April, and an observation was made on each the following October, which showed that both of them had become thinly coated with a deposit resulting from bicarbonate of lime in the water. As the only difference in the conditions was the temperature of the water, which was cold during the winter and warm during the summer, the conclusions were that the warmer the water the more rapid would be the deposit in the pipe.

Another instance of deposit from water carrying bicarbonate of lime in solution on the interior of a steel pipe was observed by the writer in connection with a 36-inch steel siphon carrying the waters from the Southern California Edison Co.'s Lytle Creek power plant from the tailrace of the plant across the canyon to the intake of the Fontana Development Co.'s canal. In 1916 the floods washed out a small section of this pipe, revealing the broken ends of the pipe still in place. On the interior was a deposit of about one-eighth of an inch in thickness over the asphaltum coating.

This pipe line was a steel riveted pipe, dipped in asphaltum according to the practice in southern California, and was installed new by the writer in the summer of 1904, and had therefore been in use for over a period of 11 years, when it was broken by the floods and observed. The deposit seemed to form over the asphaltum in exactly the same manner as it had been observed to form over the bare steel and iron surfaces in some of the pipes in the Rialto domestic water system.

The conclusion from all of the above is that deposits from water impregnated with mineral will form on the interior of any kind of a pipe, and that concrete pipes have no greater affinity in attracting such deposits than other classes of pipe.

DISCUSSION BY MR. HAZEN.

The author deserves a great deal of credit for making a large number of useful tests of the frictional resistance of water in concrete pipes. These pipes are now used to an important extent, and they are sure to find increasing application. There is a great diversity in the methods of making concrete and cement pipes, and it is not surprising to find from the tests that the coefficients vary through a wide range.

It may be suggested that the coefficients depend to some extent upon the quality of water, and not alone upon the smoothness of the pipe. In the writer's experience it has been found that reservoir waters carrying microorganisms sometimes foul pipes rapidly, and decrease carrying capacities to an important extent. This has been a vital matter with some pipe lines in this country and in England. The reduction in carrying capacity from organism may be in part, or mainly, temporary, and capacity may be restored soon after the organisms of the particular kind that cause trouble cease to be found in the water through natural causes, or by the application of copper sulphate. On the other hand, the conditions may become chronic with some reservoir waters. The matter is one that must be taken into account, and it will not do to assume that the variation in carrying capacity of concrete pipe is only due to the character of the surface of the pipe itself.

The author is to be particularly commended for making use of various approximate methods of measuring water. If he had insisted upon some one method thought to be more accurate than the others, it would have reduced the number of possible experiments. The methods used by him seem to have been sufficiently accurate. Efforts to obtain precision, while often commendable, may seriously limit the accumulation of useful data.

The writer is pleased to see that the Williams and Hazen formula still holds its own. He has used it in all his hydraulic work for 15 years, and found that, as an all-around working basis of estimate, it answers very well.

There is a distinct advantage in using only one formula, for one becomes thoroughly accustomed to it, accumulates his data in its terms, can much better judge all varying conditions, and is less likely to make errors in its application.

The formula proposed by the author for cement pipes, $V = C_s H^{0.5} d^{0.625}$, is unquestionably a good one and well adapted to the use. As a practical matter, within the ordinary range of velocities, it would not make much difference whether it or the Williams and Hazen formula were used. Only at very low or very high velocities would the difference become considerable.

The formula proposed by the author has some interesting antecedents. The number of exponential formulæ has become so great in recent years that the range of exponents is well taken up, and any exponent that may be selected will be found to have been already used by someone. Thus the Moritz formula referred to by the author at length in his paper is, in reality, the old Lampe formula, which antedates its use by Moritz by several decades. In a similar way the formula now proposed by the author for cement pipe is an old one.

In 1882, Alphonse Fteley, then city engineer of Boston, found that it best accounted for the flow of water at various depths in two sections of the Sudbury Aqueduct.[1] He wrote it: $V = 127\ R^{0.62}\ I^{0.50}$. I stood for inclination, and is equivalent to s now used. (See pipe No. 63, p. 89.)

The formula now proposed by the author was also reached by the writer in 1901 by a reconsideration of hydraulic data presented by Mr. Fenkell.[2]

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[1] Boston Water Works, Additional Supply from Sudbury River, City Document, p. 92.
[2] Jour. Assoc. Engin. Socs., vol. 26, p. 163.

The formula $V = 130\ R^{0.625}\ s^{0.50}$ was suggested for new pipe (and otherwise with varying value of the coefficient) as being more consistent and quite as accurate as some other procedures that had preceded.

Following this, a simple hydraulic-slide rule was made to calculate pipe friction by it. This permitted a great increase in rapidity of estimate. The writer used these homemade rules for a year or two in his daily work, and made a number of them for his friends. He was well satisfied with the estimates that were reached by their use. Later the interest in hydraulic-slide rules increased, and there was a demand for more of them, and it was decided to have them made in quantity. Before doing this a careful reconsideration of the exponents was made by Prof. Gardiner S. Williams, which led to the adoption of those which have since come to be associated with the Williams and Hazen formula.

This Williams and Hazen formula was intended to represent more accurately all round average waterworks conditions than the earlier one, which was identical with that one now proposed by the author, and the writer thinks that for such general use it is an improvement.

It may be pointed out that a slight variation in the exponents does not make a very great difference in ordinary estimates. Precision in the values of the exponents is not to be expected and is not necessary. It is only desirable that the formula that is used should have exponents that do not differ too widely from the actual facts.

With such a formula the important matter is a study of the coefficients to find out whether these coefficients are affected by variations in the condition and smoothness of pipe surface, and by the character of the water that flows through the pipe. At the present time study of coefficients is much more useful than further study of minute differences in exponents.

The writer is glad to see that the author is taking up his data along these lines.

DISCUSSION BY MR. LIPPINCOTT.

I have had a good deal to do with the construction of large concrete pipes, 10 to 12 feet in diameter, and the following observations may possibly be of interest:

Concrete will expand and contract with wetting and drying in much the same way that it does between heat and cold, and in order to prevent shrinkage cracks coming from drying, the concrete pipe or conduit should be kept just as moist as possible during the period in which it is curing and until it has been actually put into service. In the case of large pipe particularly this can be accomplished by putting some water into the pipe and by closing the ends of the pipe after it is built so as to prevent the circulation of dry air through the pipe. I have known concrete pipe that has developed circular shrinkage cracks which leaked badly when water was first put in and to close up after the water had been running through them for seven or eight days, I believe because of the expansion of the concrete due to its becoming saturated.

By taking pains in the manner suggested with the construction of the Los Angeles Aqueduct in the Mojave Desert, preventing the drying out of the concrete by circulation of air, mile after mile of this ditch could be built without any expansion joints and without any cracks developing.

It is suggested that concrete pipe, especially where it is to be laid on or near the surface of the ground, should where possible be built in cool weather rather than hot weather, both because of the temperature of the air and also on account of the temperature of the water. If the pipe is built in warm weather it is apt to shrink in cold weather and when cold water is run through it in such a way as to develop objectionable circular cracks.

In 1907 or 1908, at the time the Los Angeles Aqueduct was being designed, investigation was made to determine the value of the coefficient *n* in the Kutter formula in a number of the conduits of southern California. The result of these investigations was published about that time in the Engineering News over my signature. An interesting feature that developed as a result of these investigations was that the carrying capacity of the ditch and the value of the coefficient varied immensely with the question of whether the particular ditch or conduit was exposed to the rays of the sun or whether it was covered. If it were covered, the lining kept clean and smooth, but when it was exposed to the action of the sun a vegetable growth immediately set in. The growth resembled both the lichens on stone and long streaming grass. The result was that we found values of *n* of about 0.012 for ordinary covered conduits, and anywhere from 0.014 to 0.018 for these uncovered conduits.[1]

It was also observed that in ditches where the sand-box arrangements were poor that small bars of sand would follow down the conduit or even through the pipe lines and very materially, in fact totally, change the carrying capacities of the ditch. In other words, the capacity of the ditch is affected by the provisions that are made to remove sand and gravel that might otherwise enter.

My experience is that plastered conduits of any kind should if possible be covered. It has been found by experiments made, I think at the University of Michigan, that the expansion and contraction of concrete varies greatly with the richness of the mix, the richer mixes expanding more than the leaner mixes. It is customary to make the plaster richer than the main bulk of the concrete in order to get a smoother surface. The different rates of expansion and contraction of the body of the concrete and the plaster is largely responsible for the scaling of the plaster. If a plaster is desired it should have about the same ratio of sand to cement as is used in the main mix between sand and cement, and much more labor should be put on than ordinarily in producing the smooth surface. By careful forming in conduits and by troweling it is found to be possible in many cases to obtain a perfectly smooth surface without the use of plaster.

I believe that it is entirely possible to build a jointed concrete pipe that roots will not enter. A number of years ago I made an examination of the irrigation system at Rialto. This was all built of cement pipe which had been in service for a term of years, perhaps 15 or 20. The pipe ran under trees of all classes and kinds—cypress, eucalyptus, orange, and lemon trees and deciduous fruits. We cut into the pipe in many places and by means of mirrors threw the light back through the pipe so that it could be carefully examined. The pipe was clean, in every instance showing evidence of course of very careful work, but demonstrating to my mind that it was possible to build a cement pipe that roots would not enter. This was a jointed cement pipe built by Mr. Stowell, of Los Angeles. The examination was being made for purposes of reorganization.

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[1] The above experiments are described as Nos. 58, 59, and 60.

DISCUSSION BY MR. NEWELL.

With respect to the liability of concrete pipe to show decreased carrying capacity with age, my experience may be of some value. February 26, 1918, I inspected the interior of three concrete pipe lines, the R_1, D_1, and M pipes.

R_1 pipe (No. 32 and 33a).—Interior diameter 46 inches; thickness of shell 3 inches; laid in winter of 1909–10; in use 8 years. The interior for several hundred feet from the intake appeared as good as when first laid. The impressions of rivet heads in the steel forms were clearly visible in the concrete. For the first few pipe lengths from the intake there had been a very slight abrasion on the bottom of the pipe. It is over this stretch that the water flows freely when first entering the pipe and down which some gravel rolls.

D_1 pipe (No. 23 and 24a).—Interior diameter 30 inches; thickness of shell 3 inches; laid in February–March, 1910; in use 8 years. On account of the relatively small diameter of this pipe, the inspection did not cover any great distance from the intake end. A little abrasion near the bottom of the pipe was observed for the first few feet; thereafter the pipe appeared as good as new, slight irregularities in the forms being clearly indicated on the concrete.

M pipe.—Interior diameter 47 inches; thickness of shell 2½ inches; laid in winter of 1907–8; in use 10 years. On account of some seepage flow into the pipe, inspection had to be made at the outlet end. As in the other two cases, the interior seemed to be in as good a condition as when the pipe was first laid.

Considering the length of service of the three pipe lines, the fact that they are in use about 7 months each year and dry 5 months, I know of no other type of construction which seems to promise such length of service, coupled with little or no decrease in carrying capacity.

Some have felt doubt as to the permanence of the concrete pressure pipes with thin shells on account of the possible destruction of the steel due to the slow passage of water through the shell. A few days ago we had occasion to cut into the D_1 pipe for the purpose of inserting a valve, and found the steel as bright as when first laid. I am sending you a small section herewith.[1] The sample of steel was taken where the pipe is subjected to a head of slightly more than 40 feet. Samples taken from other pipe lines have uniformly given the same result, indicating that there is little likelihood of deterioration in steel in this type of pipe. I may add that in no case has our reinforced-concrete pipe, built by the wet process, shown any material defects of construction or any sign of deterioration.

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[1] To all appearances the steel is new.

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